CRSI
DESIGN HANDBOOK

Revised 1957

CONCRETE REINFORCING STEEL INSTITUTE

C.R.S.I.

FOUNDED 1924

CONCRETE REINFORCING STEEL INSTITUTE

Edward Jordan

C R S I
DESIGN HANDBOOK

REVISED 1957

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CONTENTS

PRECISION OF COMPUTATIONS FOR REINFORCED CONCRETE

If the somewhat involved mathematical methods used in rigid frame analysis lead one to believe that the design of reinforced concrete structures requires a high degree of precision, the reverse is the case. Concrete is a job-made material, and control cylinders that do not vary more than ten per cent are remarkably good. Reinforcing bars are shop-made; yet variations in strength characteristics run three to five per cent; rolled weights can vary 3½ per cent. Formwork is field-built; frequently a 2 x 8 or 2 x 10 (measuring, respectively, 7⅝ and 9½ in.) is used to form the soffit of an 8 or 10 in. beam. Bars that are held in place to an accuracy of between ⅛ and ¼ in. are extremely well placed. Two-figure accuracy is sufficient for almost all problems in reinforced concrete design.

Concrete is weak in tension; reinforcing steel is supplied to make up that deficiency; the time and effort of the designer is best spent in recognizing and providing for such tensions wherever they may exist, not in striving for a high degree of precision by carrying figures to an unmeaning number of significant places.

On the other hand, the bulk of present computing is done on a 10 in. slide rule, reading easily to three significant figures. When numbers are subtracted, significant figures are often lost. It is, therefore, recommended, more for control of the computations, for ready checking, and to keep the computer alert, rather than for any effect on the completed structure, that figures be carried to three significant places or to the extent of a 10 in. slide rule. There is no point in computing loads to a fine determination only to lose the results in a moment computation, nor is it logical to carry moments to the suggested three significant figures when the loads were guessed to one-figure precision. For that reason, the following table is suggested as a rough guide, not as any hard and fast rule, but only to give some indication of a satisfactory procedure.

RECORD VALUES TO THE FOLLOWING PRECISION:—

Loads to nearest 1 psf; 10 plf; 100 lb concentration.
Span lengths to about 0.01 ft (⅛ in. = 0.01 ft)
Total loads and reactions to 0.1 kip
Moments to nearest 0.1 kip-in., if readable
Individual bar areas to 0.01 sq in.
Concrete sizes to ½ in.
Bar spacings to ½ in. (supports are crimped at 1 in. intervals)
Effective beam depth to 0.1 in.

INTRODUCTION

The object of this handbook is to present finished designs of reinforced concrete members, giving concrete sizes and reinforcement. In the safe load tables, the need for charts and diagrams has been eliminated; the designer enters a table with load and span and immediately obtains concrete outlines and reinforcing steel. The types of construction covered are summarized in the Table of Contents, a study of which will greatly facilitate the use of these tables.

The basic theories of concrete design, a few diagrams, and some simple charts have been included for the benefit of structural engineers who prefer to make their own design computations. A brief summary of formulas for ultimate strength design is included with necessary charts.

The designs given in these tables are particularly good for preliminary estimating, for establishing sizes and clearances, and for comparing different types of construction. While they are mathematically correct for the conditions stated at the start of each table, no handbook can replace the judgment of an experienced structural engineer in selecting types of structure, proper loads, stresses, and moment factors, in eliminating eccentricities, in providing adequate stiffness, and in obtaining satisfactory and economical structures.

Designs are based entirely upon the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," with three exceptions:—(1) the maximum positive moment in end spans was taken as $wL^2/11$ (no restraint at outer end); (2) *all* bond computations are based upon deformed bars conforming to "Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (ASTM A305-53T)"; plain round bars or bars not meeting ASTM A305 can not be used satisfactorily with the values here given; (3) load capacities are extended beyond the 3:1 ratio of live to dead loads (ACI 318-56-701c).

Tabulated values for flexural members (except beams) give the maximum safe superimposed load obtained by computing the total capacity of a member as limited by flexure, shear, bond or any other consideration and deducting from the least of these the dead weight of the concrete in the member itself. In the case of beams, no useful purpose is served by deducting merely the weight of the beam and, therefore, in the beam tables only, the loads given are the *total safe loads* in pounds per lineal foot. The safe superimposed load includes live load, partitions, floor finishes, ceilings, and, in fact, everything except the dead weight of the concrete member.

Tabulated safe loads are the maximum obtainable within the stresses and factors of the 1956 ACI "Building Code Requirements for Reinforced Concrete" and should be used only by one familiar with reinforced concrete design. No increase above the tabulated values should be made.

To meet average conditions, tables are given for concrete testing 3000 psi only in standard 6 x 12 cylinders at 28 days and for deformed bars stressed 20,000 psi. Columns are worked also for the richer mixes of 3750 and 5000 psi. Weaker concretes can be worked fairly closely by direct ratio of their strength to 3000 psi, though at "balanced reinforcement" there can be quite a deviation.

U. S. Department of Commerce Simplified Practice Recommendation 26 establishes all bars as round and designates sizes by number as outlined on page 3. Throughout this book, this numbering system is used.

ASTM specification for "Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (ASTM A305-53T)" establishes the projection and spacing of deformations. Under "Building Code Requirements for Reinforced Concrete (ACI 318-56)," increased bond and diagonal tension values are permitted, provided bars meet this specification. These higher values are used throughout the book.

REFERENCES

Many useful data on reinforced concrete design are not reproduced here in their entirety. The reader is advised to procure a copy of the American Concrete Institute * "Building Code Requirements for Reinforced Concrete (ACI 318-56)," which is the recognized authority in this field; also a copy of the "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)," which supplies a great amount of information on the standard methods of delineating reinforced concrete.

He should also have a copy of the Concrete Reinforcing Steel Institute † "A Manual of Standard Practice for Reinforced Concrete Construction," 1956, which covers materials available, standard methods of fabricating, and standard practices of estimating and contracting for such items.

The Portland Cement Association ‡ issues material upon the techniques of construction and design procedures for reinforced concrete structures, most of which is available upon request.

The American Concrete Institute,* in addition to the codes and manuals described above, issues many reference books and a regular monthly publication, "Journal of the American Concrete Institute," devoted to all phases of the design of reinforced concrete structures and to better procedures for concrete proportioning, mixing, placing, and curing.

* American Concrete Institute, 18263 West McNichols Road, Detroit 19, Michigan.
† Concrete Reinforcing Steel Institute, 38 South Dearborn Street, Chicago 3, Illinois.
‡ Portland Cement Association, 33 West Grand Avenue, Chicago 10, Illinois.

WEIGHT, AREA AND PERIMETER OF INDIVIDUAL BARS

STEEL REINFORCING BARS
U. S. Department of Commerce
Simplified Practice Recommendation 26

Bar Numbers and Weights

Bar #[a]	Weight Per Foot (lb)	Bar #[a]	Weight Per Foot (lb)
2[b]	0.167	7	2.044
3	0.376	8	2.670
4	0.668	9[c]	3.400
5	1.043	10[c]	4.303
6	1.502	11[c]	5.313

Data on Standard Deformed Bars

Obsolete Bar Designation (Size, in.)	Bar Designation	Unit Weight Per Foot	Nominal Dimensions—Round Sections		
			Diameter	Cross-Sectional Area	Perimeter
Rounds:	#	lb	in.	sq in.	in.
¼	2[b]	0.167	0.250	0.05	0.786
⅜	3	0.376	0.375	0.11	1.178
½	4	0.668	0.500	0.20	1.571
⅝	5	1.043	0.625	0.31	1.963
¾	6	1.502	0.750	0.44	2.356
⅞	7	2.044	0.875	0.60	2.749
1	8	2.670	1.000	0.79	3.142
Squares:					
1	9[c]	3.400	1.128	1.00	3.544
1⅛	10[c]	4.303	1.270	1.27	3.990
1¼	11[c]	5.313	1.410	1.56	4.430

Dimensional Requirements for Deformed Steel Bars for Concrete Reinforcement
ASTM Serial Designation A305 *

Deformed Bar Designation Number[a]	Deformation Requirements		
	Max Avg Spacing (in.)	Min Height (in.)	Max Gap (in.) Chord of 12½ Per Cent of Nominal Perimeter
3	0.262	0.015	0.143
4	0.350	0.020	0.191
5	0.437	0.028	0.239
6	0.525	0.038	0.286
7	0.612	0.044	0.334
8	0.700	0.050	0.383
9[c]	0.790	0.056	0.431
10[c]	0.889	0.064	0.487
11[c]	0.987	0.071	0.540

[a] Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.

[b] Bar #2 in plain rounds only.

[c] Bars of designation #9, #10 and #11 correspond to former 1-in. square, 1⅛-in. square and 1¼-in. square sizes and are equivalent to those former standard bar sizes in weight and nominal cross-sectional areas.

* Weights, diameters, areas and perimeters of ASTM A305 are given above in "Data on Standard Deformed Bars."

AREAS OF VARIOUS COMBINATIONS OF BARS AND MINIMUM WEB WIDTHS FOR BEAMS

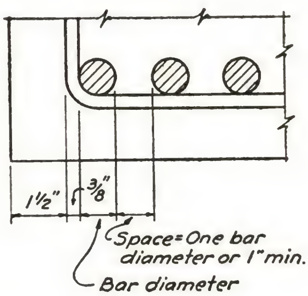
For table of areas of various combinations of bars, see pages 5, 6 and 7.

This table gives all practicable combinations of bars of equal diameters or differing by one or two sizes to produce any desired steel area, A_s.

This table also gives the minimum width of beam web that will properly cover the bars placed in a single layer, on the basis of $1\frac{1}{2}$ in. protection over $\frac{3}{8}$ stirrup legs, with spaces between the bars equal to one bar diameter or one inch. (See figure below.) Beam widths are given in multiples of one-tenth inch. When bars are of different sizes, smaller bars are placed on outside.

Example:—A_s = 2.38 sq in. can be obtained with 4-#6 and 2-#5 bars, requiring a beam width of 13 in. or better, or with 4-#7 bars in a beam width of 10½ in.

Aggregate should be chosen with maximum size three-quarters of the clear space between bars.



The following table gives minimum beam widths in multiples of one-quarter inch for various numbers of *equal* sized bars spaced as on the figure above:-

MINIMUM BEAM WIDTHS—ACI CODE

Size of Bars	Number of Bars in Single Layer of Reinforcement							Add for Each Added Bar
	2	3	4	5	6	7	8	
#4	5¾	7¼	8¾	10¼	11¾	13¼	14¾	1½
#5	6	7¾	9¼	11	12½	14¼	15¾	1⅝
#6	6¼	8	9¾	11½	13¼	15	16¾	1¾
#7	6½	8½	10¼	12¼	14	16	17¾	1⅞
#8	6¾	8¾	10¾	12¾	14¾	16¾	18¾	2
#9	7¼	9½	11¾	14	16¼	18½	20¾	2¼
#10	7¾	10¼	12¾	15¼	17¾	20¼	23	2⅝
#11	8	11	13¾	16½	19½	22¼	25	2⅞

Table shows minimum beam widths when stirrups are used.

If no stirrups are required, deduct three-quarters of an inch from figures shown.

For additional bars, add dimension in last column for each added bar.

For bars of different sizes, determine from table the beam width for smaller size bars, and then add last column figure for each larger bar used, or see pages 5 to 7.

AREAS OF VARIOUS COMBINATIONS OF BARS AND MINIMUM WEB WIDTHS FOR BEAMS

A_s (sq in.)	Bar Combination Quant	Size	Quant	Size	Min Web Width (in.)
0.11	1	#3	.....	....	4.2
0.20	1	#4	.....	....	4.3
0.31	1	#5	.....	....	4.4
0.31	1	#4	1	#3	5.7
0.40	2	#4	.....	....	5.8
0.42	1	#4	2	#3	7.0
0.44	1	#6	.....	....	4.5
0.51	1	#5	1	#4	5.9
0.51	2	#4	1	#3	7.2
0.53	1	#4	3	#3	8.4
0.60	1	#7	.....	....	4.7
0.60	3	#4	.....	....	7.3
0.62	2	#5	.....	....	6.0
0.62	2	#4	2	#3	8.5
0.64	1	#4	4	#3	9.8
0.64	1	#6	1	#4	6.0
0.71	1	#5	2	#4	7.4
0.71	3	#4	1	#3	8.7
0.73	2	#4	3	#3	9.9
0.75	1	#6	1	#5	6.2
0.75	1	#4	5	#3	11.2
0.79	1	#8	.....	....	4.8
0.80	4	#4	.....	....	8.8
0.82	2	#5	1	#4	7.5
0.82	3	#4	2	#3	10.0
0.84	2	#4	4	#3	11.3
0.84	1	#6	2	#4	7.5
0.88	2	#6	.....	....	6.3
0.91	1	#5	3	#4	8.9
0.91	1	#7	1	#5	6.3
0.91	4	#4	1	#3	10.2
0.93	3	#5	.....	....	7.7
0.93	3	#4	3	#3	11.4
0.95	2	#4	5	#3	12.7
1.00	1	#9	.....	....	4.9
1.00	5	#4	.....	....	10.3
1.02	2	#5	2	#4	9.0
1.02	4	#4	2	#3	11.5
1.04	1	#7	1	#6	6.4
1.04	3	#4	4	#3	12.8
1.04	1	#6	3	#4	9.0
1.06	1	#6	2	#5	7.8
1.08	2	#6	1	#4	7.8
1.11	1	#5	4	#4	10.4
1.11	5	#4	1	#3	11.7

A_s (sq in.)	Bar Combination Quant	Size	Quant	Size	Min Web Width (in.)
1.13	3	#5	1	#4	9.2
1.13	4	#4	3	#3	12.9
1.15	3	#4	5	#3	14.2
1.19	2	#6	1	#5	7.9
1.20	2	#7	.....	....	6.5
1.22	2	#5	3	#4	10.5
1.22	1	#7	2	#5	7.9
1.22	5	#4	2	#3	13.0
1.23	1	#8	1	#6	6.5
1.24	4	#5	.....	....	9.3
1.24	1	#6	4	#4	10.5
1.24	4	#4	4	#3	14.3
1.27	1	#10	.....	....	5.1
1.28	2	#6	2	#4	9.3
1.31	1	#5	5	#4	11.9
1.32	3	#6	.....	....	8.0
1.33	3	#5	2	#4	10.7
1.33	5	#4	3	#3	14.4
1.35	4	#4	5	#3	15.7
1.37	1	#6	3	#5	9.4
1.39	1	#8	1	#7	6.7
1.42	2	#5	4	#4	12.0
1.44	4	#5	1	#4	10.8
1.44	1	#6	5	#4	12.0
1.44	5	#4	4	#3	15.8
1.48	1	#7	2	#6	8.2
1.48	2	#6	3	#4	10.8
1.50	2	#6	2	#5	9.5
1.51	2	#7	1	#5	8.2
1.52	3	#6	1	#4	9.5
1.53	1	#7	3	#5	9.5
1.53	3	#5	3	#4	12.2
1.55	5	#5	.....	....	10.9
1.55	5	#4	5	#3	17.2
1.56	1	#11	.....	....	5.2
1.58	2	#8	.....	....	6.8
1.60	1	#9	1	#7	6.9
1.62	2	#5	5	#4	13.5
1.63	3	#6	1	#5	9.7
1.64	4	#5	2	#4	12.3
1.64	2	#7	1	#6	8.3
1.67	1	#8	2	#6	8.3
1.68	1	#6	4	#5	11.0
1.68	2	#6	4	#4	12.3
1.72	3	#6	2	#4	11.0

A_s (sq in.)	Bar Combination Quant	Size	Quant	Size	Min Web Width (in.)
1.73	3	#5	4	#4	13.7
1.75	5	#5	1	#4	12.4
1.76	4	#6	.....	....	9.8
1.79	1	#9	1	#8	7.0
1.80	3	#7	.....	....	8.4
1.81	2	#6	3	#5	11.2
1.82	2	#7	2	#5	9.8
1.84	1	#7	4	#5	11.2
1.84	4	#5	3	#4	13.8
1.88	2	#6	5	#4	13.8
1.92	1	#7	3	#6	9.9
1.92	3	#6	3	#4	12.5
1.93	3	#5	5	#4	15.2
1.94	3	#6	2	#5	11.3
1.95	5	#5	2	#4	13.9
1.96	4	#6	1	#4	11.3
1.99	1	#8	2	#7	8.5
1.99	1	#6	5	#5	12.7
2.00	2	#9	.....	....	7.2
2.02	2	#8	1	#6	8.5
2.04	4	#5	4	#4	15.3
2.06	1	#10	1	#8	7.2
2.07	4	#6	1	#5	11.4
2.08	2	#7	2	#6	10.0
2.11	1	#8	3	#6	10.0
2.11	3	#7	1	#5	10.0
2.12	2	#6	4	#5	12.8
2.12	3	#6	4	#4	14.0
2.13	2	#7	3	#5	11.4
2.15	1	#7	5	#5	12.8
2.15	5	#5	3	#4	15.4
2.16	4	#6	2	#4	12.8
2.18	2	#8	1	#7	8.7
2.20	1	#9	2	#7	8.7
2.20	5	#6	.....	....	11.5
2.24	4	#5	5	#4	16.8
2.24	3	#7	1	#6	10.2
2.25	3	#6	3	#5	12.9
2.27	1	#10	1	#9	7.4
2.32	3	#6	5	#4	15.5
2.35	5	#5	4	#4	16.9
2.36	1	#7	4	#6	11.7
2.36	4	#6	3	#4	14.3
2.37	3	#8	.....	....	8.8
2.38	4	#6	2	#5	13.0

AREAS OF VARIOUS COMBINATIONS OF BARS AND MINIMUM WEB WIDTHS FOR BEAMS

A_s (sq in.)	Bar Combination				Min Web Width (in.)	A_s (sq in.)	Bar Combination				Min Web Width (in.)	A_s (sq in.)	Bar Combination				Min Web Width (in.)
	Quant	Size	Quant	Size			Quant	Size	Quant	Size			Quant	Size	Quant	Size	
2.40	4	#7	.....	....	10.3	3.12	2	#11	.....	....	8.0	3.98	2	#8	4	#7	14.3
2.40	5	#6	1	#4	13.0	3.12	3	#7	3	#6	13.7	4.00	4	#9	.....	....	11.7
2.42	3	#7	2	#5	11.7	3.13	5	#6	3	#5	16.4	4.00	1	#9	5	#7	14.4
2.43	2	#6	5	#5	14.4	3.16	4	#8	.....	....	10.8	4.00	3	#7	5	#6	17.2
2.44	2	#7	4	#5	13.0	3.19	1	#8	4	#7	12.3	4.04	4	#8	2	#6	14.3
2.46	2	#8	2	#6	10.3	3.20	2	#9	2	#7	11.1	4.10	1	#11	2	#10	10.4
2.51	5	#6	1	#5	13.2	3.20	5	#6	5	#4	19.0	4.12	2	#10	2	#8	11.9
2.52	2	#7	3	#6	11.8	3.25	3	#8	2	#6	12.3	4.12	2	#11	1	#9	10.4
2.54	2	#10	.....	....	7.6	3.27	1	#10	2	#9	9.7	4.13	3	#8	4	#6	15.8
2.55	5	#5	5	#4	18.4	3.28	4	#7	2	#6	13.8	4.16	1	#9	4	#8	13.1
2.55	1	#8	4	#6	11.8	3.31	4	#6	5	#5	17.9	4.16	4	#7	4	#6	17.3
2.56	3	#6	4	#5	14.5	3.31	5	#7	1	#5	13.8	4.17	3	#8	3	#7	14.4
2.56	1	#11	1	#9	7.6	3.33	2	#10	1	#8	9.7	4.20	3	#9	2	#7	13.3
2.56	4	#6	4	#4	15.8	3.33	4	#7	3	#5	15.2	4.24	5	#7	4	#5	18.7
2.58	1	#9	2	#8	9.1	3.34	2	#8	4	#6	13.8	4.27	1	#10	3	#9	12.0
2.59	1	#8	3	#7	10.4	3.35	3	#7	5	#5	16.5	4.32	5	#7	3	#6	17.4
2.60	2	#9	1	#7	9.1	3.37	1	#9	3	#8	11.1	4.36	4	#8	2	#7	14.5
2.60	5	#6	2	#4	14.5	3.38	2	#8	3	#7	12.4	4.37	2	#9	3	#8	13.3
2.68	3	#7	2	#6	11.9	3.40	1	#9	4	#7	12.4	4.39	5	#8	1	#6	14.5
2.69	4	#6	3	#5	14.7	3.40	2	#7	5	#6	15.3	4.39	2	#11	1	#10	10.6
2.71	4	#7	1	#5	11.9	3.44	5	#7	1	#6	13.9	4.40	2	#9	4	#7	14.8
2.73	3	#7	3	#5	13.3	3.44	5	#6	4	#5	18.0	4.43	1	#10	4	#8	13.3
2.75	2	#7	5	#5	14.7	3.54	2	#10	1	#9	9.9	4.48	4	#8	3	#6	16.0
2.76	4	#6	5	#4	17.3	3.56	1	#11	2	#9	10.0	4.54	2	#10	2	#9	12.3
2.78	2	#8	2	#7	10.5	3.56	3	#7	4	#6	15.4	4.55	5	#8	1	#7	14.7
2.79	2	#9	1	#8	9.2	3.57	3	#8	2	#7	12.5	4.55	5	#7	5	#5	20.3
2.80	1	#9	3	#7	10.6	3.58	2	#9	2	#8	11.3	4.56	1	#11	3	#9	12.2
2.80	1	#7	5	#6	13.4	3.60	3	#9	1	#7	11.4	4.57	3	#8	5	#6	17.5
2.80	5	#6	3	#4	16.0	3.60	4	#8	1	#6	12.5	4.58	3	#9	2	#8	13.6
2.81	3	#8	1	#6	10.5	3.62	5	#7	2	#5	15.4	4.58	2	#8	5	#7	16.2
2.82	5	#6	2	#5	14.8	3.64	1	#10	3	#8	11.3	4.60	3	#10	1	#8	12.3
2.83	1	#11	1	#10	7.8	3.64	4	#7	4	#5	16.8	4.60	4	#9	1	#7	13.6
2.84	4	#7	1	#6	12.0	3.69	3	#8	3	#6	14.0	4.60	4	#7	5	#6	19.0
2.85	1	#10	2	#8	9.3	3.72	4	#7	3	#6	15.5	4.68	3	#11	.....	....	10.9
2.87	3	#6	5	#5	16.2	3.75	5	#6	5	#5	19.7	4.76	5	#7	4	#6	19.2
2.88	2	#9	2	#6	10.8	3.76	4	#8	1	#7	12.7	4.77	3	#8	4	#7	16.3
2.96	2	#7	4	#6	13.5	3.78	2	#8	5	#6	15.5	4.79	4	#9	1	#8	13.8
2.97	3	#8	1	#7	10.7	3.79	3	#9	1	#8	11.5	4.80	3	#9	3	#7	15.2
2.99	1	#8	5	#6	13.5	3.79	1	#8	5	#7	14.2	4.81	3	#10	1	#9	12.5
3.00	4	#6	4	#5	16.3	3.80	2	#9	3	#7	12.9	4.83	5	#8	2	#6	16.3
3.00	3	#9	.....	....	9.4	3.81	3	#10	.....	....	10.2	4.91	2	#10	3	#8	13.8
3.00	5	#7	.....	....	12.2	3.88	5	#7	2	#6	15.7	4.92	4	#8	4	#6	17.8
3.00	5	#6	4	#4	17.5	3.93	5	#7	3	#5	17.0	4.95	1	#9	5	#8	15.1
3.02	4	#7	2	#5	13.5	3.95	5	#8	.....	....	12.8	4.96	4	#8	3	#7	16.4
3.04	3	#7	4	#5	14.9	3.95	4	#7	5	#5	18.4	5.00	5	#9	.....	....	14.0

AREAS OF VARIOUS COMBINATIONS OF BARS AND MINIMUM WEB WIDTHS FOR BEAMS

A_s (sq in.)	Bar Combination				Min Web Width (in.)	A_s (sq in.)	Bar Combination				Min Web Width (in.)	A_s (sq in.)	Bar Combination				Min Web Width (in.)
	Quant	Size	Quant	Size			Quant	Size	Quant	Size			Quant	Size	Quant	Size	
5.00	2	#9	5	#7	16.6	6.24	4	#11	.....	....	13.7	8.08	4	#10	3	#9	19.6
5.08	4	#10	.....	....	12.7	6.27	1	#10	5	#9	16.5	8.12	2	#11	5	#9	19.6
5.12	2	#11	2	#9	12.7	6.35	5	#10	.....	....	15.2	8.16	5	#9	4	#8	22.1
5.15	5	#8	2	#7	16.5	6.35	5	#8	4	#7	20.3	8.20	2	#11	4	#10	18.3
5.16	2	#9	4	#8	15.3	6.37	4	#9	3	#8	17.8	8.24	4	#10	4	#8	21.0
5.20	4	#9	2	#7	15.4	6.40	4	#9	4	#7	19.3	8.24	4	#11	2	#9	18.5
5.20	5	#7	5	#6	20.9	6.49	2	#10	5	#8	17.9	8.35	5	#10	2	#9	19.9
5.22	1	#10	5	#8	15.3	6.54	2	#10	4	#9	16.8	8.49	3	#11	3	#10	18.6
5.27	5	#8	3	#6	18.0	6.56	1	#11	5	#9	16.8	8.68	3	#11	4	#9	20.2
5.27	1	#10	4	#9	14.2	6.58	5	#9	2	#8	18.1	8.72	5	#10	3	#8	21.5
5.36	4	#8	5	#6	19.5	6.64	1	#11	4	#10	15.5	8.78	4	#11	2	#10	18.9
5.37	1	#11	3	#10	13.0	6.66	4	#10	2	#8	17.0	8.80	5	#11	1	#9	18.9
5.37	3	#9	3	#8	15.5	6.68	3	#11	2	#9	15.6	8.81	3	#10	5	#9	21.6
5.37	3	#8	5	#7	18.2	6.80	5	#9	3	#7	19.7	8.95	5	#9	5	#8	24.1
5.39	3	#10	2	#8	14.4	6.81	3	#10	3	#9	17.1	9.03	4	#10	5	#8	23.0
5.40	3	#9	4	#7	17.1	6.93	2	#11	3	#10	15.8	9.07	5	#11	1	#10	19.1
5.54	2	#10	3	#9	14.5	6.95	3	#9	5	#8	19.6	9.08	4	#10	4	#9	21.9
5.56	4	#8	4	#7	18.3	6.95	5	#8	5	#7	22.2	9.24	4	#11	3	#9	20.7
5.56	1	#11	4	#9	14.5	6.97	3	#10	4	#8	18.4	9.35	5	#10	3	#9	22.1
5.58	4	#9	2	#8	15.8	7.00	4	#9	5	#7	21.2	9.47	2	#11	5	#10	20.9
5.60	5	#9	1	#7	15.9	7.08	4	#10	2	#9	17.3	9.51	5	#10	4	#8	23.5
5.66	2	#11	2	#10	13.2	7.12	2	#11	4	#9	17.3	9.68	3	#11	5	#9	22.4
5.68	3	#11	1	#9	13.2	7.14	5	#10	1	#8	17.4	9.76	3	#11	4	#10	21.1
5.70	2	#10	4	#8	15.9	7.16	4	#9	4	#8	19.8	9.80	5	#11	2	#9	21.3
5.71	5	#8	4	#6	19.8	7.22	3	#11	2	#10	16.1	10.05	4	#11	3	#10	21.4
5.75	5	#8	3	#7	18.4	7.24	4	#11	1	#9	16.1	10.08	4	#10	5	#9	24.1
5.79	5	#9	1	#8	16.0	7.35	5	#10	1	#9	17.6	10.24	4	#11	4	#9	23.0
5.80	4	#9	3	#7	17.4	7.37	5	#9	3	#8	20.1	10.30	5	#10	5	#8	25.5
5.81	3	#10	2	#9	14.8	7.40	5	#9	4	#7	21.6	10.34	5	#11	2	#10	21.7
5.87	4	#10	1	#8	14.8	7.45	4	#10	3	#8	19.0	10.35	5	#10	4	#9	24.4
5.95	2	#9	5	#8	17.3	7.51	4	#11	1	#10	16.3	10.80	5	#11	3	#9	23.5
5.95	3	#11	1	#10	13.5	7.54	2	#10	5	#9	19.0	11.03	3	#11	5	#10	23.7
6.00	3	#9	5	#7	18.9	7.68	3	#11	3	#9	17.9	11.24	4	#11	5	#9	25.2
6.08	4	#10	1	#9	15.0	7.76	3	#10	5	#8	20.4	11.32	4	#11	4	#10	24.0
6.12	2	#11	3	#9	15.1	7.80	5	#11	.....	....	16.5	11.35	5	#10	5	#9	26.7
6.15	5	#8	5	#6	21.5	7.81	3	#10	4	#9	19.3	11.61	5	#11	3	#10	24.2
6.16	3	#9	4	#8	17.6	7.91	1	#11	5	#10	18.0	11.80	5	#11	4	#9	25.8
6.16	4	#8	5	#7	20.2	7.93	5	#10	2	#8	19.5	12.59	4	#11	5	#10	26.5
6.18	3	#10	3	#8	16.4	7.95	4	#9	5	#8	21.8	12.80	5	#11	5	#9	28.1
6.20	5	#9	2	#7	17.8	8.00	5	#9	5	#7	23.5	12.88	5	#11	4	#10	26.8

PERIMETERS FOR VARIOUS COMBINATIONS OF BARS

		0		1	2	3	4	5		1	2	3	4	5
1		1.6		2.7	3.9	5.1	6.3	7.5						
2		3.1		4.3	5.5	6.7	7.9	9.0						
3	#4	4.7	#3	5.9	7.1	8.2	9.4	10.6						
4		6.3		7.5	8.6	9.8	11.0	12.2						
5		7.9		9.0	10.2	11.4	12.6	13.7						
1		2.0		3.5	5.1	6.7	8.2	9.8						
2		3.9		5.5	7.1	8.6	10.2	11.8						
3	#5	5.9	#4	7.5	9.0	10.6	12.2	13.7						
4		7.9		9.4	11.0	12.6	14.1	15.7						
5		9.8		11.4	13.0	14.5	16.1	17.7						
1		2.4		4.3	6.3	8.2	10.2	12.2		3.9	5.5	7.1	8.6	10.2
2		4.7		6.7	8.6	10.6	12.6	14.5		6.3	7.9	9.4	11.0	12.6
3	#6	7.1	#5	9.0	11.0	13.0	14.9	16.9	#4	8.6	10.2	11.8	13.4	14.9
4		9.4		11.4	13.4	15.3	17.3	19.2		11.0	12.6	14.1	15.7	17.3
5		11.8		13.7	15.7	17.7	19.6	21.6		13.4	14.9	16.5	18.1	19.6
1		2.7		5.1	7.5	9.8	12.2	14.5		4.7	6.7	8.6	10.6	12.6
2		5.5		7.9	10.2	12.6	14.9	17.3		7.5	9.4	11.4	13.3	15.3
3	#7	8.2	#6	10.6	13.0	15.3	17.7	20.0	#5	10.2	12.2	14.1	16.1	18.1
4		11.0		13.4	15.7	18.1	20.4	22.8		13.0	14.9	16.9	18.8	20.8
5		13.7		16.1	18.5	20.8	23.2	25.5		15.7	17.7	19.6	21.6	23.6
1		3.1		5.9	8.6	11.4	14.1	16.9		5.5	7.9	10.2	12.6	14.9
2		6.3		9.0	11.8	14.5	17.3	20.0		8.6	11.0	13.4	15.7	18.1
3	#8	9.4	#7	12.2	14.9	17.7	20.4	23.2	#6	11.8	14.1	16.5	18.9	21.2
4		12.6		15.3	18.1	20.8	23.6	26.3		14.9	17.3	19.6	22.0	24.3
5		15.7		18.5	21.2	24.0	26.7	29.5		18.1	20.4	22.8	25.1	27.5
1		3.5		6.7	9.8	13.0	16.1	19.3		6.3	9.0	11.8	14.5	17.3
2		7.1		10.2	13.4	16.5	19.7	22.8		9.8	12.6	15.3	18.1	20.8
3	#9	10.6	#8	13.8	16.9	20.1	23.2	26.3	#7	13.4	16.1	18.9	21.6	24.4
4		14.2		17.3	20.5	23.6	26.7	29.9		16.9	19.7	22.4	25.2	27.9
5		17.7		20.9	24.0	27.1	30.3	33.4		20.5	23.2	26.0	28.7	31.5
1		4.0		7.5	11.1	14.6	18.2	21.7		7.1	10.3	13.4	16.6	19.7
2		8.0		11.5	15.1	18.6	22.2	25.7		11.1	14.3	17.4	20.5	23.7
3	#10	12.0	#9	15.5	19.1	22.6	26.1	29.7	#8	15.1	18.3	21.4	24.5	27.7
4		16.0		19.5	23.0	26.6	30.1	33.7		19.1	22.2	25.4	28.5	31.7
5		20.0		23.5	27.0	30.6	34.1	37.7		23.1	26.2	29.4	32.5	35.7
1		4.4		8.4	12.4	16.4	20.4	24.4		8.0	11.5	15.1	18.6	22.2
2		8.9		12.9	16.8	20.8	24.8	28.8		12.4	15.9	19.5	23.0	26.6
3	#11	13.3	#10	17.3	21.3	25.3	29.3	33.2	#9	16.8	20.4	23.9	27.5	31.0
4		17.7		21.7	25.7	29.7	33.7	37.7		21.3	24.8	28.4	31.9	35.4
5		22.2		26.1	30.1	34.1	38.1	42.1		25.7	29.2	32.8	36.3	39.9

The column headed 0 contains the total perimeter for bars of the size given in the second column, the number of bars, from 1 to 5, being specified in the first column.

Columns headed 1 2 3 4 5 add to Column 0 the perimeter of the number of bars called for across the top of the table of the size given just to the left of the number of bars.

Examples:—Perimeter of three #4 bars is found on Line 3 in Column 0 as 4.7 in.

Perimeter of three #4 plus four #3 bars is found on Line 3 in Column headed 4 as 9.4 in.

Perimeter of five #11 plus five #9 bars is found in the last line, last column as 39.9 in.

AREAS AND PERIMETERS OF BARS FOR SECTION OF SLAB ONE-FOOT WIDE

The first table gives the cross-sectional area per foot width of slab for various combinations of equal and unequal size bars. The spacing given is center to center of adjacent bars in inches, whether bars are of the same or different size. When different bar sizes are combined, large and small bars alternate.

The next table gives the total perimeter of bars per foot width of slab for various bar spacings. The spacing selected for entering the table should be that of the bars actually available for bond, disregarding any bars that have been bent out of the plane of stress.

Example in the use of Area Table on page 11. An area of 1.18 sq in. per foot width of slab is required. The column headed "#6 + #7" shows that 1.19 sq in. can be obtained with consecutive bars 5¼ * in. c/c. The required area can also be obtained with #5 + #6 @ 3¾ * = 1.20 sq in. However, #6 + #7 @ 5 * = 1.25 sq in. has the advantage of spacing in even inches and will be used.

Example in the use of Perimeter Table on page 12. To obtain the perimeter for bond, assume first that this is a single span, so that alternate #6 bars would be bent up and bond figured on #7 @ 10 in. Enter the table and in column headed "#7" at a spacing of 10 in. find a perimeter of 3.3 in.

If this were a continuous span, the #7 bars would be bent up and bond for negative moment would be figured on two sets of #7 @ 10 in., which is equivalent to #7 @ 5 in. In column headed "#7" at a spacing of 5 in. find 6.6 in. Bond for the positive moment bars is figured on #6 @ 10 in. = 2.8 in.

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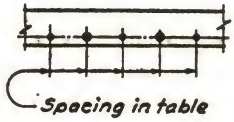
* According to "Building Code Requirements for Reinforced Concrete (ACI 318-56)," the spacing between two adjacent bars must not exceed three times the slab thickness, *t*.

AREAS OF BARS FOR SECTION OF SLAB ONE-FOOT WIDE



	Spacing* (in.)	Combinations of Bar Nos								
		#2+#2	#2+#3	#3+#3	#3+#4	#4+#4	#4+#5	#5+#5	#5+#6	#6+#6
1	2	0.30	0.48	0.66	0.93	1.20	1.53	1.86	2.25	2.64
2	2¼	0.27	0.42	0.59	0.82	1.07	1.36	1.65	2.00	2.35
3	2½	0.24	0.38	0.53	0.74	0.96	1.22	1.49	1.80	2.11
4	2¾	0.22	0.35	0.48	0.68	0.87	1.12	1.35	1.64	1.92
5	3	0.20	0.32	0.44	0.62	0.80	1.02	1.24	1.50	1.76
6	3¼	0.18	0.29	0.41	0.57	0.74	0.94	1.14	1.38	1.62
7	3½	0.17	0.28	0.38	0.53	0.69	0.87	1.06	1.28	1.51
8	3¾	0.16	0.26	0.35	0.50	0.64	0.82	0.99	1.20	1.41
9	4	0.15	0.24	0.33	0.47	0.60	0.77	0.93	1.13	1.32
10	4¼	0.14	0.23	0.31	0.44	0.56	0.72	0.88	1.06	1.24
11	4½	0.13	0.22	0.29	0.42	0.53	0.68	0.83	1.00	1.17
12	4¾	0.13	0.20	0.28	0.39	0.51	0.64	0.78	0.95	1.11
13	5	0.12	0.19	0.26	0.37	0.48	0.61	0.74	0.90	1.06
14	5¼	0.11	0.18	0.25	0.36	0.46	0.58	0.71	0.85	1.01
15	5½	0.11	0.17	0.24	0.34	0.44	0.56	0.68	0.82	0.96
16	5¾	0.10	0.16	0.23	0.32	0.42	0.53	0.65	0.78	0.92
17	6	0.10	0.16	0.22	0.31	0.40	0.51	0.62	0.75	0.88
18	6½	0.09	0.15	0.20	0.29	0.37	0.47	0.57	0.70	0.81
19	7	0.09	0.14	0.19	0.27	0.34	0.44	0.53	0.65	0.75
20	7½	0.08	0.13	0.18	0.25	0.32	0.41	0.50	0.60	0.70
21	8	0.08	0.13	0.17	0.24	0.30	0.38	0.47	0.56	0.66
22	8½	0.07	0.11	0.16	0.22	0.28	0.36	0.44	0.53	0.62
23	9	0.07	0.11	0.15	0.21	0.27	0.34	0.41	0.50	0.59
24	9½	0.06	0.10	0.14	0.20	0.25	0.32	0.39	0.48	0.56
25	10	0.06	0.10	0.13	0.19	0.24	0.31	0.37	0.45	0.53
26	10½	0.06	0.09	0.13	0.18	0.23	0.29	0.35	0.43	0.50
27	11	0.05	0.09	0.12	0.17	0.22	0.28	0.34	0.41	0.48
28	11½	0.05	0.08	0.11	0.16	0.21	0.27	0.32	0.39	0.46
29	12	0.05	0.08	0.11	0.16	0.20	0.26	0.31	0.38	0.44
30	13	....	....	0.10	0.14	0.18	0.24	0.29	0.35	0.41
31	14	....	....	0.09	0.13	0.17	0.22	0.27	0.33	0.38
32	15	....	....	0.09	0.13	0.16	0.21	0.25	0.30	0.35

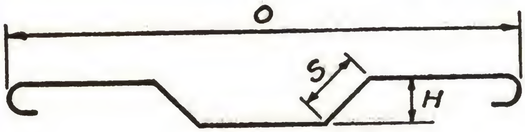
* According to "Building Code Requirements for Reinforced Concrete (ACI 318-56)," the spacing between two adjacent bars must not exceed three times the slab thickness, *t*.

AREAS OF BARS FOR SECTION OF SLAB ONE-FOOT WIDE

Combinations of Bar Nos										
#6+#7	#7+#7	#7+#8	#8+#8	#8+#9	#9+#9	#9+#10	#10+#10	#10+#11	#11+#11	
										1
2.77	3.20									2
2.50	2.88	3.34	3.79							3
2.27	2.62	3.03	3.45							4
2.08	2.40	2.78	3.16	3.58	4.00					5
1.92	2.22	2.57	2.92	3.31	3.69					6
1.78	2.06	2.38	2.71	3.06	3.43	3.89	4.36			7
1.66	1.92	2.22	2.53	2.86	3.20	3.63	4.06	4.53	4.99	8
1.56	1.80	2.09	2.37	2.69	3.00	3.41	3.81	4.25	4.68	9
1.47	1.69	1.97	2.23	2.53	2.82	3.20	3.59	3.99	4.40	10
1.39	1.60	1.85	2.11	2.38	2.67	3.02	3.39	3.77	4.16	11
1.32	1.52	1.76	2.00	2.26	2.53	2.86	3.21	3.57	3.94	12
1.25	1.44	1.67	1.90	2.15	2.40	2.72	3.05	3.39	3.74	13
1.19	1.37	1.59	1.81	2.04	2.29	2.59	2.90	3.23	3.57	14
1.13	1.31	1.51	1.72	1.95	2.18	2.48	2.77	3.09	3.40	15
1.09	1.25	1.45	1.65	1.86	2.09	2.37	2.65	2.96	3.26	16
1.04	1.20	1.39	1.58	1.79	2.00	2.27	2.54	2.83	3.12	17
0.96	1.11	1.28	1.46	1.65	1.85	2.09	2.35	2.61	2.88	18
0.89	1.03	1.19	1.35	1.54	1.71	1.95	2.18	2.43	2.67	19
0.83	0.96	1.11	1.26	1.43	1.60	1.82	2.03	2.27	2.50	20
0.78	0.90	1.04	1.19	1.34	1.50	1.70	1.91	2.12	2.34	21
0.73	0.85	0.98	1.12	1.27	1.41	1.61	1.79	2.00	2.20	22
0.69	0.80	0.93	1.05	1.20	1.33	1.52	1.69	1.89	2.08	23
0.66	0.76	0.88	1.00	1.13	1.26	1.43	1.60	1.79	1.97	24
0.63	0.72	0.84	0.95	1.08	1.20	1.36	1.52	1.70	1.87	25
0.60	0.69	0.80	0.90	1.02	1.14	1.30	1.45	1.62	1.78	26
0.57	0.65	0.76	0.86	0.98	1.09	1.24	1.39	1.55	1.70	27
0.55	0.63	0.73	0.82	0.93	1.04	1.19	1.33	1.48	1.63	28
0.52	0.60	0.70	0.79	0.90	1.00	1.14	1.27	1.42	1.56	29
0.48	0.55	0.64	0.73	0.83	0.92	1.05	1.17	1.31	1.44	30
0.45	0.51	0.60	0.68	0.77	0.86	0.98	1.09	1.22	1.34	31
0.42	0.48	0.56	0.63	0.72	0.80	0.91	1.02	1.14	1.25	32

SLANTS AND INCREMENTS FOR 45° BAR BENDS *

See also page 85.



O = Overall Bar Dimension
H = Height of Bend
S = Slant = 1.414 *H* to Nearest ½ Inch
I = Increment = *S–H*

Height *H* †	Slant *S*	Increment 2 Slants 2*I*	Height *H* †	Slant *S*	Increment 2 Slants 2*I*	Height *H* †	Slant *S*	Increment 2 Slants 2*I*
			1-1	1-6½	11	3-1	4-4½	2-7
			1-2	1-8	1-0	3-2	4-5½	2-7
			1-3	1-9	1-0	3-3	4-7	2-8
2	3	2	1-4	1-10½	1-1	3-4	4-8½	2-9
2½	3½	2	1-5	2-0	1-2	3-5	4-10	2-10
3	4	2	1-6	2-1½	1-3	3-6	4-11½	2-11
3½	5	3	1-7	2-3	1-4	3-7	5-1	3-0
4	5½	3	1-8	2-4	1-4	3-8	5-2	3-0
4½	6½	4	1-9	2-5½	1-5	3-9	5-3½	3-1
5	7	4	1-10	2-7	1-6	3-10	5-5	3-2
5½	7½	4	1-11	2-8½	1-7	3-11	5-6½	3-3
6	8½	5	2-0	2-10	1-8	4-0	5-8	3-4
6½	9	5	2-1	2-11½	1-9	4-1	5-9½	3-5
7	10	6	2-2	3-1	1-10	4-2	5-10½	3-5
7½	10½	6	2-3	3-2	1-10	4-3	6-0	3-6
8	11½	7	2-4	3-3½	1-11	4-4	6-1½	3-7
8½	1-0	7	2-5	3-5	2-0	4-5	6-3	3-8
9	1-0½	7	2-6	3-6½	2-1	4-6	6-4½	3-9
9½	1-1½	8	2-7	3-8	2-2	4-7	6-6	3-10
10	1-2	8	2-8	3-9	2-2	4-8	6-7	3-10
10½	1-3	9	2-9	3-10½	2-3	4-9	6-8½	3-11
11	1-3½	9	2-10	4-0	2-4	4-10	6-10	4-0
11½	1-4	9	2-11	4-1½	2-5	4-11	6-11½	4-1
1-0	1-5	10	3-0	4-3	2-6	5-0	7-1	4-2

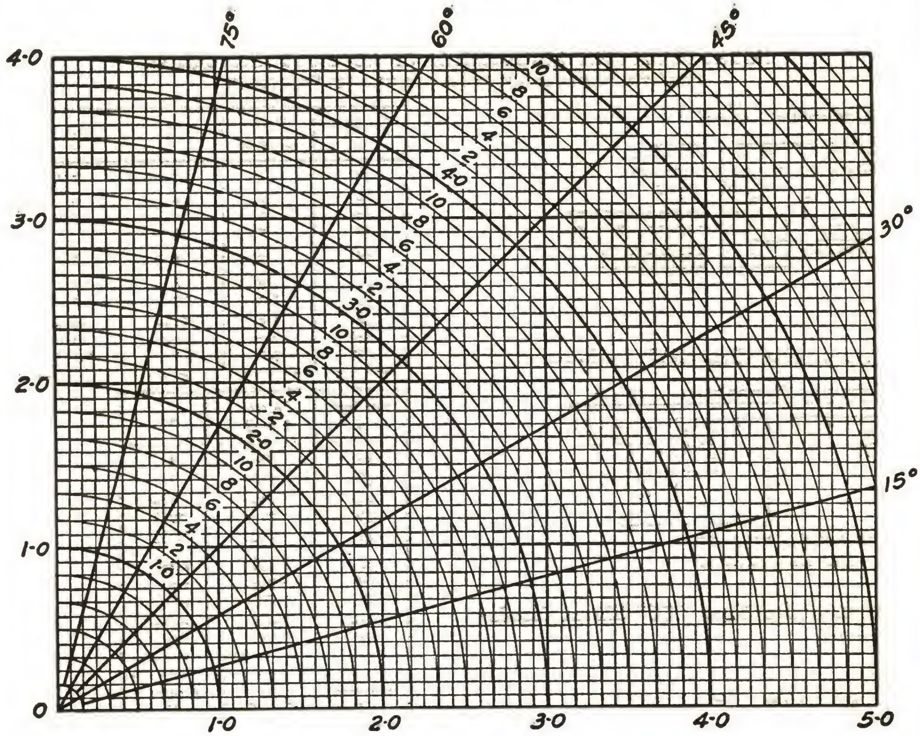
Increment For 2 Slants = 2 × (*S–H*)
Length of Truss Bars = 0 + 2*I* + Hooks.
All Dimensions Are Out To Out of Bar.
Scheduled length of bar is sum of the detail dimensions.

* From "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

† H = out-to-out vertical drop of truss bar = out-to-out of concrete, less following where applicable:—(1) fireproofing at top, (2) fireproofing at bottom, (3) tie diameter top, (4) stirrup diameter bottom, (5) diameter of cross-over bars, (6) allowance for bottom layer of bars and clearance between bars.

Fireproofing is usually ¾″ at top and ¾″ at bottom for slabs and joists, 1½″ top and 1½″ bottom for beams, to outer side of stirrups.

SLANT DIAGRAM *



To determine the slant length enter the diagram with the length and height of bend and at the intersection read the slant length on the curved line.

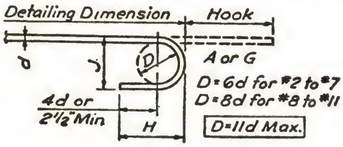
* From "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

STANDARD HOOKS

These details for standard hooks are taken from the ACI "Manual of Standard Practice for Detailing Reinforced Concrete Structures."

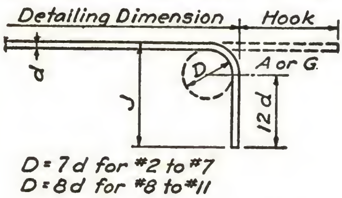
HOOKS WHICH MEET REQUIREMENTS OF ACI 318-56 AND CAN BE READILY FABRICATED WITH STANDARD EQUIPMENT

Recommended Sizes 180° Hook:—



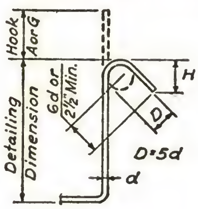
Bar Size d	Hook A or G	J	Approx. H
#2	4	2	3½
#3	5	3	4
#4	6	4	4½
#5	7	5	5
#6	8	6	6
#7	10	7	7
#8	1-1	10	9
#9	1-3	11¼	10¼
#10	1-5	1-0½	11¼
#11	1-7	1-2	1-0¾

Recommended Sizes 90° Hook:—



Bar Size d	Hook A or G	J
#2	3½	4
#3	5½	6
#4	7½	8¼
#5	9	10¼
#6	10½	1-0½
#7	1-0½	1-2½
#8	1-2½	1-5
#9	1-4½	1-7
#10	1-6½	1-9½
#11	1-8½	2-0

Recommended Sizes 135° Stirrup Hook:—

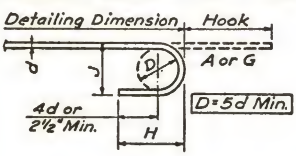


Note:—When supporting bars are used, stirrup hooks may be bent to the diameter of the supporting bars.

Bar Size d	Hook A or G	Approx. H
#2	3½	2¼
#3	4	2½
#4	5	3
#5	6	3¾
#6	7	4½

MINIMUM HOOKS THAT CAN BE FABRICATED WITH STANDARD EQUIPMENT

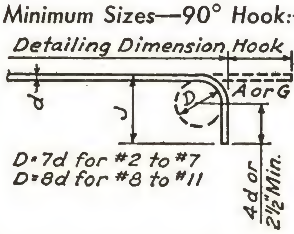
Minimum Sizes—180° Hook:—



Note:—This table to be used only for special conditions, where hooks smaller than recommended sizes are necessary. Not appropriate for hard grades of steel.

Bar Size d	Hook A or G	J	Approx. H
#2	4	1¾	3½
#3	5	2¾	4
#4	5	3½	4¼
#5	6	4¼	4¾
#6	7	5¼	5¾
#7	9	6	6½
#8	10	7	7½
#9	11	8	8½
#10	1-1	9	9½
#11	1-2	10	10½

Minimum Sizes—90° Hook:—



Bar Size d	Hook A or G	J
#2	3	3½
#3	3½	4
#4	3½	5
#5	4	5½
#6	4½	6½
#7	5½	7½
#8	6½	9
#9	7½	10
#10	8½	11½
#11	9	1-0½

DATA ON AS&W WIRE GAUGES USED IN WELDED WIRE FABRIC*

AS&W Wire Gauge Numbers	Diameter (in.)	Area (sq in.)	Weight (lb/ft)
0000	0.3938	0.12180	0.4136
000	0.3625	0.10321	0.3505
00	0.3310	0.086049	0.2922
0	0.3065	0.073782	0.2506
1	0.2830	0.062902	0.2136
2	0.2625	0.054119	0.1838
3	0.2437	0.046645	0.1584
4	0.2253	0.039867	0.1354
5	0.2070	0.033654	0.1143
6	0.1920	0.028953	0.09832
7	0.1770	0.024606	0.08356
8	0.1620	0.020612	0.07000
9	0.1483	0.017273	0.05866
10	0.1350	0.014314	0.04861
11 †	0.1205	0.011404	0.03873
12 †	0.1055	0.0087417	0.02969
13 †	0.0915	0.0065755	0.02233
14 †	0.0800	0.0050266	0.01707
15 †	0.0720	0.0040715	0.01383
16 †	0.0625	0.0030680	0.01042

WELDING RANGE OF TRANSVERSE WIRE GAUGES FOR LONGITUDINAL WIRE OF GIVEN GAUGE *

Gauge Longitudinal Wire	Gauge Transverse Wire Range	
	Maximum	Minimum
0000	000	4
000	000	4
00	000	5
0	000	7
1	000	7
2	000	9
3	000	9
4	000	9
5	00	10
6	0	10
7	1	11
8	3	12
9	4	12
10	6	12
11	7	16
12	8	16
13	12	16
14	12	16
15	12	16
16	12	16

* All mesh tables from "Design Manual for Welded Wire Fabric" of Wire Reinforcement Institute, Inc., 1955.

† Fabric in which both longitudinal and transverse wires are No. 11 gauge or lighter is furnished galvanized only.

COMMON STYLES OF WELDED WIRE FABRIC

ONE-WAY TYPES

Style Designation	Spacing of Wires (in.)		Size of Wires AS&W Gauge		Sectional Area * (sq in. per ft)		Weight (lb per 100 sq ft)
	Longit.	Trans.	Longit.	Trans.	Longit.	Trans.	
2 x 12—0/6	2	12	0	6	.443	.029	166
2 x 16—0/6	2	16	0	6	.443	.022	163
2 x 16—1/7	2	16	1	7	.377	.018	140
2 x 16—2/8	2	16	2	8	.325	.015	119
2 x 16—3/8	2	16	3	8	.280	.015	104
2 x 16—4/9	2	16	4	9	.239	.013	89
2 x 16—5/10	2	16	5	10	.202	.011	75
2 x 16—6/10	2	16	6	10	.174	.011	65
2 x 16—7/11	2	16	7	11	.148	.009	55
3 x 16—2/8	3	16	2	8	.216	.015	83
3 x 16—3/8	3	16	3	8	.187	.015	72
3 x 16—4/9	3	16	4	9	.159	.013	61
4 x 16—3/8	4	16	3	8	.140	.015	56
4 x 16—4/9	4	16	4	9	.120	.013	48
4 x 16—5/10	4	16	5	10	.101	.011	40
4 x 16—6/10	4	16	6	10	.087	.011	35
4 x 16—7/11	4	16	7	11	.074	.009	30
4 x 16—8/12	4	16	8	12	.062	.007	25
4 x 16—9/12	4	16	9	12	.052	.007	21
4 x 12—4/9	4	12	4	9	.120	.017	49
4 x 12—5/7	4	12	5	7	.101	.025	45
4 x 12—5/10	4	12	5	10	.101	.014	42
4 x 12—6/10	4	12	6	10	.087	.014	36
4 x 12—7/11	4	12	7	11	.074	.011	31
4 x 12—8/12	4	12	8	12	.062	.009	26
4 x 12—9/12	4	12	9	12	.052	.009	22
4 x 8—7/11	4	8	7	11	.074	.017	33
4 x 8—8/12	4	8	8	12	.062	.013	27
4 x 8—9/12	4	8	9	12	.052	.013	23
4 x 8—10/12	4	8	10	12	.043	.013	20
6 x 12—00/4	6	12	00	4	.172	.040	78
6 x 12—0/0	6	12	0	0	.148	.074	81
6 x 12—0/3	6	12	0	3	.148	.047	72
6 x 12—1/1	6	12	1	1	.126	.063	69
6 x 12—1/4	6	12	1	4	.126	.040	61
6 x 12—2/2	6	12	2	2	.108	.054	59
6 x 12—2/5	6	12	2	5	.108	.034	52
6 x 12—3/3	6	12	3	3	.093	.047	51
6 x 12—4/4	6	12	4	4	.080	.040	44
6 x 12—6/6	6	12	6	6	.058	.029	32

The above styles are used mostly in building construction.

Although the above styles are termed "one-way" fabrics—since in each case, the transverse wires are of minimum permissible size and have maximum permissible spacing—actually they have some transverse reinforcing effectiveness by virtue of the amount of transverse steel provided.

* "Building Code Requirements for Reinforced Concrete (ACI 318)" (306b) permits a maximum tensile stress of 30,000 psi in one-way slabs on spans of not more than 12 ft, so that the steel area if wire fabric is used in one-way slab tables under 12-ft span can be ⅔ of that given for bars (pp. 123-130).

COMMON STYLES OF WELDED WIRE FABRIC

TWO-WAY TYPES

Style Designation	Spacing of Wires (in.)		Size of Wires AS&W Gauge		Sectional Area † (sq in. per ft)		Weight (lb per 100 sq ft)
	Longit.	Trans.	Longit.	Trans.	Longit.	Trans.	
2 x 2—10/10	2	2	10	10	.086	.086	60
2 x 2—12/12 *	2	2	12	12	.052	.052	37
2 x 2—14/14 *	2	2	14	14	.030	.030	21
3 x 3—8/8	3	3	8	8	.082	.082	58
3 x 3—10/10	3	3	10	10	.057	.057	41
3 x 3—12/12 *	3	3	12	12	.035	.035	25
3 x 3—14/14 *	3	3	14	14	.020	.020	14
4 x 4—4/4	4	4	4	4	.120	.120	85
4 x 4—6/6	4	4	6	6	.087	.087	62
4 x 4—8/8	4	4	8	8	.062	.062	44
4 x 4—10/10	4	4	10	10	.043	.043	31
4 x 4—12/12 *	4	4	12	12	.026	.026	19
4 x 4—13/13 *	4	4	13	13	.020	.020	14
6 x 6—0/0	6	6	0	0	.148	.148	107
6 x 6—1/1	6	6	1	1	.126	.126	91
6 x 6—2/2	6	6	2	2	.108	.108	78
6 x 6—3/3	6	6	3	3	.093	.093	68
6 x 6—4/4	6	6	4	4	.080	.080	58
6 x 6—4/6	6	6	4	6	.080	.058	50
6 x 6—5/5	6	6	5	5	.067	.067	49
6 x 6—6/6	6	6	6	6	.058	.058	42
6 x 6—7/7	6	6	7	7	.049	.049	36
6 x 6—8/8	6	6	8	8	.041	.041	30
6 x 6—9/9	6	6	9	9	.035	.035	25
6 x 6—10/10	6	6	10	10	.029	.029	21

A two-way fabric—for a given size of longitudinal wires—is any style in which the sectional area of transverse steel is greater than the minimum required for proper fabrication by reason of the transverse wires either having a spacing which is less than the permissible maximum, or being of larger size than the permissible minimum.

Two-way fabrics are not necessarily limited to styles in which longitudinal and transverse wires both have the same size and spacing as indicated in the above table.

* Usually furnished only in galvanized wire.

† "Building Code Requirements for Reinforced Concrete (ACI 318)" (306b) permits a maximum tensile stress of 30,000 psi in one-way slabs on spans of not more than 12 ft, so that the steel area if wire fabric is used in one-way slab tables under 12-ft span can be ⅔ of that given for bars (pp. 123-130).

FORMULAS, TABLES AND DIAGRAMS FOR REINFORCED CONCRETE DESIGN

NOMENCLATURE

A — Area of a section or transformed area of a reinforced section.

A_c — Area of core of a spirally-reinforced column measured to the outside of the spiral; net area of concrete section of a composite column.

A_g — Gross area of spirally-reinforced or tied columns; total area of concrete encasement of a combination column.

A_r — Area of steel or cast iron core in a composite column; area of steel core in a combination column.

A_s — Effective cross-sectional area of available reinforcing steel.

A'_s — Area of compressive reinforcement in flexural members.

A_t — Area of temperature reinforcement.

A_v — Total area of web reinforcement in tension within a distance of s (measured in a direction parallel to that of the main reinforcement) or the total area of all bars bent up in any one plane.

a — Base length of shear diagram in inches.

α — Angle between inclined web bars and axis of beam.

B — A factor in column design; $B = CD$.

b — Width of rectangular flexural member or width of flanges for T and I sections.

b' — Width of web in T and I flexural members.

C — Ratio of allowable concrete stress, f_a, in axially-loaded columns to allowable fiber stress for concrete in flexure; also resultant of compressive stress.

C_c — Resultant of compression in concrete only.

C_s — Resultant of compression in compressive reinforcement only.

c — Effective support size.

D — $\dfrac{t^2}{2^2K}$ = a factor in column design varying from 3 to 9, where t is the overall depth of column section and K is the least radius of gyration of the section; the diameter of spiral; deflection produced by a test load of a flexural member relative to the ends of a span.

d — The depth from compression face of a beam or slab to the centroid of longitudinal tensile reinforcement; also the diameter of a reinforcing bar.

d' — Distance from extreme compressive fiber to centroid of compressive reinforcement.

E_c — The modulus of elasticity of concrete in compression.

E_s — The modulus of elasticity of reinforcing steel.

e — Eccentricity of the axial load on a column measured from the gravity axis.

e' — Eccentricity measured from tensile steel axis.

F_a — Nominal allowable axial unit stress $(0.225 f'_c + f_s p_g)$ for spiral columns and 0.8 of this value for tied columns.

F_b — Allowable bending unit stress that would be permitted if bending stress only existed.

f_a — Nominal axial unit stress = axial load divided by area of member, A_g.

f_b — Bending unit stress (actual) = bending moment divided by section modulus of member.

f_c — Computed stress in extreme fiber on compressive side of a reinforced concrete flexural member or computed concrete fiber stress in an eccentrically-loaded column, where the ratio of $e/t > 2/3$.

f'_c — Ultimate compressive strength of concrete at 28 days, unless otherwise specified.

f_r — Allowable unit stress in the metal core of a composite column.

f'_r — Allowable unit stress on unencased steel columns and pipe columns.

f_s — Computed stress in tensile reinforcement of beams (for allowable working stresses see page 32). Nominal allowable stress in vertical column reinforcement (for allowable working stresses see page 32).

f'_s — Computed stress in compressive reinforcement of beams and eccentrically loaded columns; also useful limit stress of spiral reinforcement, to be taken as 40,000 psi for hot-rolled bars of intermediate grade, 50,000 psi for bars of hard grade, and 60,000 psi for cold-drawn wire.

f_v — Tensile unit stress in web reinforcement.

g — Ratio depth of steel to depth of concrete in eccentrically-loaded columns.

H — Unsupported length of column.

h — Any vertical height or distance.

I — Moment of inertia of a section about the neutral axis for bending.

j — Ratio of distance between centroid of compression and centroid of tension to the depth d.

K — Stiffness factor in frame design, i.e., the moment of inertia divided by the length.

K_c — Radius of gyration of concrete in concrete-filled pipe columns.

NOMENCLATURE

K_s Radius of gyration of a metal pipe section (in pipe columns).
k Ratio of distance from extreme compressive fiber to neutral axis to the depth d.
L Span length of flat slab center-to-center of columns in the direction of which bending is considered; also length of embedment to develop bond stress; span of member under load test (shorter span of flat slabs and of floors supported on four sides).
l Span length of slab or beam.
l' Clear span for positive moment and shear and the average of the two adjacent clear spans for negative moment.
$\left.\begin{array}{l} l'' \\ l''' \end{array}\right\}$ Corresponding spans adjacent to l'.
M External bending moment in lb-in.
M_c Resisting moment as determined by concrete.
M_o Sum of the positive and the average negative bending moment at the critical design sections of a flat slab panel.
M_r Resisting moment of internal stresses in lb-in.
M_s Resisting moment as determined by reinforcing steel.
N Axial load applied to reinforced concrete column; also number of stirrups.
n Ratio of modulus of elasticity (E_s) to that of concrete (E_c) $= \dfrac{E_s}{E_c} = \dfrac{30{,}000}{f'_c}$.
$N.A.$ Neutral axis.
P Total allowable axial load on a column whose length does not exceed ten times its least cross-sectional dimension; also external concentric load on footings or piles; also concentrated load on flexural members.
P' Total allowable axial load on a long column.
p Ratio of tensile reinforcement in beams $= A_s/bd$.
p' Ratio of volume of spiral reinforcement to the volume of the concrete core (out-to-out of spirals) of a spirally-reinforced concrete column; also ratio of compressive reinforcement in beams $= A'_s/bd$.
p_a Net active soil pressure in footing design.
p_g Ratio of the effective cross-sectional area of vertical reinforcement to the gross area, A_g.
p_t Total soil pressure in footing design.
pcf Pounds per cubic foot.
plf Pounds per lineal foot.
psf Pounds per square foot.
psi Pounds per square inch.
R M/bd^2, the constant for flexural computations; also radius of circular column cap.
r Radius of circular column in inches
s Spacing of stirrups or bent bars in a direction parallel to that of the main reinforcement; also spacing of main bars in solid slabs.
Σ_o Sum of perimeters of bars in one set.
T Resultant of tensile stresses.
t Overall depth of rectangular column section or the diameter of a round column; also depth of flange of a tee beam or thickness of a slab; thickness or depth of a member under load test.
t_1 Thickness in inches of slab without drop panels, or through drop panel if any.
t_2 Thickness in inches of slab with drop panels at points beyond the drop panel.
u Bond stress per unit of surface area of bar.
V Total shear; also total vertical force.
V' Excess of the total shear over that permitted on the concrete.
v Shearing unit stress.
v' Shearing stress taken by web reinforcement.
v_c Allowable intensity of diagonal tension (shear) resisted by concrete.
W Total load (wL)
w Uniformly distributed load per unit of length of beam or per unit of area of slab.
z Distance between extreme fiber and resultant of compressive stresses; also intensity of horizontal shear per lineal inch.

Nomenclature for ultimate strength design are defined on page 28.

Symbols

$=$ Sign of equality; i.e., equal to.
$>$ More than, indicating that the value to the left is greater than that to the right of the symbol.
$<$ Less than, indicating that the value to the left is smaller than that to the right of the symbol.
$\geqq$ More than or equal to.
$\leqq$ Less than or equal to.
Σ Sum of, indicating the adding up of terms.

FORMULAS, TABLES AND DIAGRAMS FOR REINFORCED CONCRETE DESIGN

Although the purpose of this book is to provide finished designs giving concrete outlines and reinforcing steel for different loading conditions, it may be helpful to have the formulas for various stress computations of reinforced concrete grouped together in one place for easy reference.

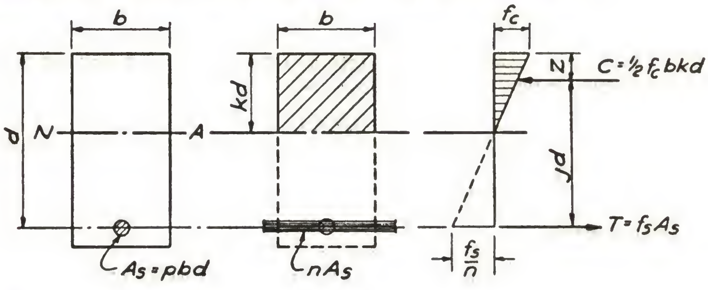
References in this section marked "ACI" are to "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

Transformed Areas

In general, it will be found with practice that it is simpler to use the method of "transformed areas" than to depend upon formulas. This method simply replaces the reinforcement with $n\left(=\frac{E_s}{E_c}\right)$ times as much concrete on the tension side of a member or $(n - 1)$ times as much on the compression side. It omits any tension concrete to produce the equivalent of a homogeneous member whose neutral axis (under flexure without direct stress) is at the centroid of the transformed area. If stresses vary as the distance from the neutral axis, all values can be determined by simple geometrical relationships.

FORMULAS—WORKING STRESS METHOD

1. Rectangular Beams with Tension Reinforcement.*



f_s = tensile unit stress in steel.
f_c = compressive unit stress in extreme fiber of concrete.
E_s = modulus of elasticity of steel.
E_c = modulus of elasticity of concrete.

$n = \frac{E_s}{E_c}$

M = moment of resistance or bending moment in general.
b = breadth of beam.
d = depth of beam to center of steel.

* For applications to specific numerical examples, see pages 116, 132, 211, 219, 224.

FORMULAS, TABLES AND DIAGRAMS

A_s = cross-sectional area of tension steel reinforcement.
k = ratio of depth of neutral axis to depth d.
j = ratio of lever arm of resisting couple to depth d.
z = depth from compression face to resultant of the compressive stresses.
$jd = d - z$ = arm of resisting couple.
p = steel ratio $\frac{A_s}{bd}$

$$M = Tjd = A_s f_s jd = (f_s pj)bd^2 = Cjd = (\tfrac{1}{2}f_c kj)bd^2 = Rbd^2$$

$$R_s = f_s pj \qquad R_c = \tfrac{1}{2}f_c kj$$

$$k = \sqrt{2pn + (pn)^2} - pn \qquad z = \frac{kd}{3} \qquad j = 1 - \frac{k}{3}$$

$$k = \frac{1}{1 + \frac{f_s}{nf_c}} \qquad p = \frac{f_c}{2f_s}k$$

$$f_s = \frac{M}{A_s jd} \qquad f_c = \frac{2M}{kjbd^2}$$

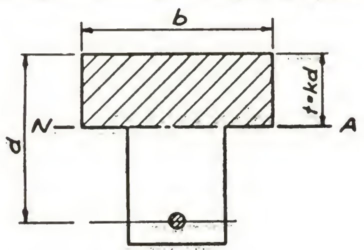
Balanced Reinforcement:—$p = \dfrac{1}{\frac{2f_s}{f_c}\left(\frac{f_s}{nf_c} + 1\right)}$

For tables and diagrams giving values of these constants for different stresses and grades of concrete, see pages 33, 34, 36, 37, 38.

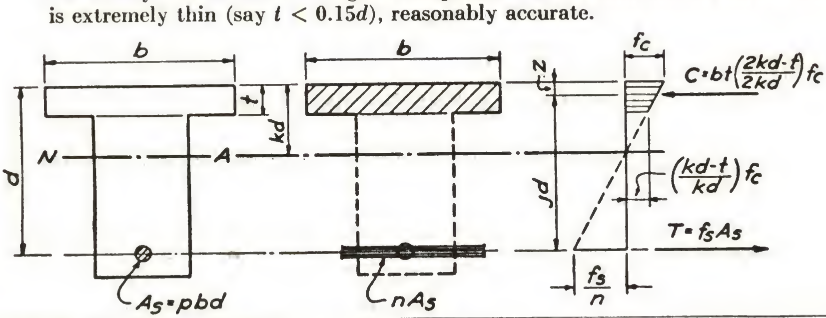
2. Tee Beams with Tension Reinforcement.*

ACI 705 limits b in symmetrical beams to $\frac{L}{4}$ and the projection either side of the stem to $8t$, or one-half the distance to the next beam. In one-sided beams, it limits the projection beyond the stem to $\frac{L}{12}$, $6t$, or one-half the distance to the next beam.

A. Flange thickness down to neutral axis:—
This becomes a rectangular beam (see Item 1) of width b.



B. Shallow flange, neglecting stem:—
It is always conservative to neglect compression in the stem and, unless the flange is extremely thin (say $t < 0.15d$), reasonably accurate.



* For application to specific numerical examples, see pages 140, 156, 172, 211.

FORMULAS, TABLES AND DIAGRAMS

Symbols have same meaning as for rectangular beams, Item 1 above; also

t = flange thickness.

$$z = \frac{t}{3}\frac{(3kd - 2t)}{(2kd - t)}$$

$$jd = d - z = d - \frac{t}{3}\frac{(3kd - 2t)}{(2kd - t)}$$

$$M_s = Tjd = A_s f_s jd = f_s jpbd^2 = \left(\frac{f_s}{n}\right)pnjbd^2 = C_s\left(\frac{f_s}{n}\right)bd^2 = R_s bd^2$$

where $C_s = pnj$; $R_s = f_s pj$

$$M_c = Cjd = btj\left(\frac{2kd - t}{2k}\right)f_c = \frac{f_c j \frac{t}{d}}{2k}\left(2k - \frac{t}{d}\right)bd^2 = R_c bd^2 =$$

$$f_c\left(1 - \frac{t}{2kd}\right)\frac{t}{d}jbd^2 = C_c f_c bd^2$$

where $C_c = \left(1 - \frac{t}{2kd}\right)\frac{t}{d}j$; $R_c = f_c j\frac{t}{d}\left(\frac{2k - \frac{t}{d}}{2k}\right)$

$$k = \frac{1}{\left(\frac{f_s}{nf_c} + 1\right)}$$

$$kd = \frac{2ndA_s + bt^2}{2nA_s + 2bt}; \qquad k = \frac{2pn + \left(\frac{t}{d}\right)^2}{2pn + 2\left(\frac{t}{d}\right)}$$

$$f_c = \frac{Mkd}{bt\left(kd - \frac{t}{2}\right)jd} = \frac{f_s k}{n(1 - k)} \qquad f_s = \frac{M}{A_s jd} = \frac{f_c n(1 - k)}{k}$$

$$j = \frac{6 - 6\left(\frac{t}{d}\right) + 2\left(\frac{t}{d}\right)^2 + \frac{\left(\frac{t}{d}\right)^3}{2pn}}{6 - 3\left(\frac{t}{d}\right)}$$

C. Shallow flange, including stem:—

No formulas are presented here because this refinement is seldom necessary, no diagrams or tables are readily available, and the method of transformed areas is the simplest attack on the problem.

D. Doubly Reinforced Tee Beams:—

It is possible to reinforce a tee beam for compression, but, additional flange width being far more economical, compressive reinforcement would only be used when both the width and depth of beam are severely limited by space considerations. This relatively unusual case is best solved by the method of transformed areas.

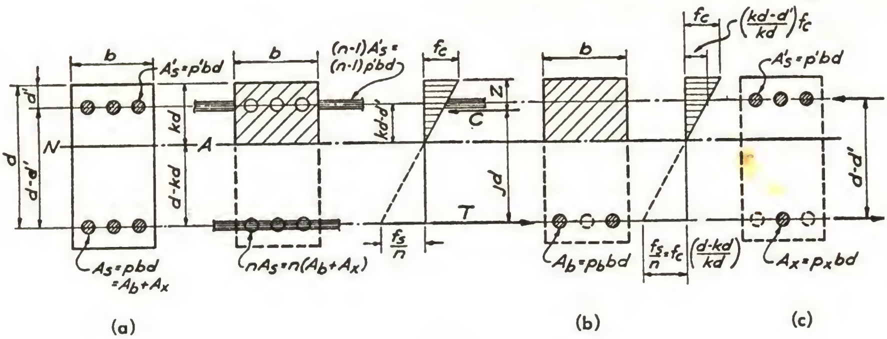
For tables and diagrams giving values of these constants for different stresses and grades of concrete, see pages 35, 39, 40, 41.

3. Beams Reinforced for Both Tension and Compression.*

Under the elastic theory, the doubly reinforced beam (a) (figure on page 25), can be thought of as a rectangular beam (b) with balanced tension reinforcement plus a supplementary internal couple (c) of the compression in the top steel and the stress in the additional tension steel, the excess area over that required for balanced reinforcement. The diagrams usually provided for doubly reinforced beams are drawn with the added compression based upon nA_s instead of the value $(n - 1)A_s$ required to compensate for the displaced concrete.

* For applications to examples, see "Negative Moment" on pages 157, 173 and "Negative Flexure" on pages 220 and 225.

FORMULAS, TABLES AND DIAGRAMS



For balanced reinforcement as in (b), R_b is easily computed or available from tables on page 33. For the couple in (c), the resisting moment, M_x, is the product of the compressive area $(n-1)p'bd$, the stress intensity $\left(\frac{kd-d'}{kd}\right)f_c$ and the arm $(d-d')$. From this, $R_x = \frac{M_x}{bd^2} = (n-1)p'\left(1-\frac{1}{k}\frac{d'}{d}\right)f_c\left(1-\frac{d'}{d}\right)$ and $R = R_b + R_x$. The extra tension steel, A_x, being further from the neutral axis and not displacing any flexurally stressed concrete, will be less in amount than A'_s, so $p_x = \left(\frac{kd-d'}{d-kd}\right)\left(\frac{n-1}{n}\right)p'$. If p_b is the ratio for balanced reinforcement, $p = p_b + p_x$. For any given set of stresses and $\frac{d'}{d}$ ratio, the relation of R to p and p' will plot a straight line, so curves are easily constructed for any set of values (see page 45).

The following formulas can be used, but the procedure just described should be simpler:—

$$k = \sqrt{2n\left(p + p'\frac{d'}{d}\right) + n^2(p+p')^2} - n(p+p')$$

$$z = \frac{\frac{1}{3}k^3d + 2p'nd'\left(k - \frac{d'}{d}\right)}{k^2 + 2p'n\left(k - \frac{d'}{d}\right)} \qquad jd = (d - z)$$

$$f_s = \frac{M}{pjbd^2} = \frac{nf_c(1-k)}{k}$$

$$f'_s = \frac{nf_c\left(k - \frac{d'}{d}\right)}{k}$$

$$f_c = \frac{6M}{bd^2\left[3k - k^2 + \frac{6p'n}{k}\left(k - \frac{d'}{d}\right)\left(1 - \frac{d'}{d}\right)\right]}$$

Note:—To approximate the effect of creep, the effectiveness of compressive reinforcement is taken as double that indicated by the elastic theory, providing that the unit stress does not then exceed that permitted in tension, f_s = 20,000 psi (ACI 706b).

For tables and diagrams giving values of these constants for different stresses and grades of concrete, see pages 42, 43, 44.

FORMULAS, TABLES AND DIAGRAMS

4. Web Reinforcement and Bond.*

Intensity of web shear:—$v = \dfrac{V}{b'jd}$ (ACI 801a)

Point where no web reinforcement is required:—$a = \left(\dfrac{v - v_c}{v}\right)$ times distance to zero shear. Then add stirrups, spaced not over $d/2$, for a distance d beyond this point (ACI 801d); for isolated continuous beams or frames, carry web reinforcement $L/16$ or d past extreme position of point of inflection to carry 2/3 total shear on any section.

Stress in a series of parallel bent bars (ACI 804d):—$f_v = \dfrac{V's}{A_s jd(\sin \alpha + \cos \alpha)}$

Stress in vertical stirrups:—$f_v = \dfrac{V's}{A_v jd}$

Area of stirrups in distance s:—$A_v = \dfrac{V's}{f_v jd} = \dfrac{v'bs}{f_v}$

Spacing of stirrups, where N equals the number of stirrups required:—

$$s = \frac{f_v jdA_v}{V'}; \quad s = a\left(\frac{\sqrt{N} - \sqrt{N - \frac{1}{2}}}{\sqrt{N}}\right);$$

$$a\left(\frac{\sqrt{N} - \sqrt{N - 1\frac{1}{2}}}{\sqrt{N}}\right); \quad a\left(\frac{\sqrt{N} - \sqrt{N - 2\frac{1}{2}}}{\sqrt{N}}\right); \text{ etc.}$$

Bond:—$u = \dfrac{V}{\Sigma ojd} = \dfrac{vb}{\Sigma o}$

Length to develop bar:—$L = \dfrac{f_s D}{4u}$

5. Columns (Concentric Load).

A. Tied Columns † (ACI 1104a):—

$P = A_g(0.18f'_c + 0.8f_s p_g)$.

A_g = gross area of column.

f'_c = compressive strength of concrete.

f_s = nominal allowable stress in vertical reinforcement (40 per cent of minimum specified yield point, i.e., 16,000 psi for intermediate grade and 20,000 psi for rail or hard grade steel).

p_g = ratio of area of vertical reinforcement to gross area, A_g.

B. Spirally Reinforced Columns ‡ (ACI 1103a):—

$P = A_g(0.225 f'_c + f_s p_g)$

where the symbols have the same meaning as in 5-A above.

Spiral Reinforcement:—§

$$p' = 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f'_s} \qquad \text{(ACI 1103d)}$$

where:—

p' = ratio of volume of spiral reinforcement to volume of concrete core, out-to-out of spirals.

f'_s = useful limit stress of spiral reinforcement; 40,000 psi for intermediate grade hot rolled rods; 50,000 psi for hard grade; and 60,000 psi for cold drawn wire.

C. Long Columns (H > 10t):—

$P' = P\left(1.3 - 0.03\dfrac{H}{t}\right)$, where:—

* See further explanation on pages 85-91 and examples on pages 212, 220, and 225.

† For applications to specific numerical examples, see page 235.

‡ For applications to specific numerical examples, see pages 250 and 251.

§ For applications to specific numerical examples, see page 251.

|| For table of reduction factors, see page 233; for numerical example, page 235.

FORMULAS, TABLES AND DIAGRAMS

P = allowable axial load on a short column.
H = unsupported height of column, $> 10t$.
t = side of column.
P' = maximum allowable axial load on a column where $H > 10t$.

This same reduction formula shall apply to an eccentrically loaded column if P is the allowable eccentrically applied load on a short column.

In long columns subjected to definite bending stress, $\frac{H}{t} \lesseqgtr 20$.

6. Eccentrically Loaded Columns:— *

A. Axial load and bending in one principal plane, $e \lesseqgtr \frac{2}{3}t$ (uncracked section):—

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} \lesseqgtr 1 \qquad \text{(ACI 1109a)} \qquad \left(C = \frac{P/A}{0.45f'_c}\right)$$

$$P = N\left[1 + \frac{CDe}{t}\right] = N\left[1 + \frac{Be}{t}\right]$$

$$N = \frac{P}{\left[1 + \frac{CDe}{t}\right]} = \frac{P}{\left[1 + \frac{Be}{t}\right]}$$

For D, see page 277.
For B, see pages 358-360.

B. Axial load and bending on both principal axes, $e \lesseqgtr \frac{2}{3}t$ in each direction (uncracked section):—

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_b} + \frac{f_{by}}{F_b} \lesseqgtr 1 \qquad \text{(ACI 1109b)}$$

C. Axial load and bending, $e > \frac{2}{3}t$ (cracked section):—

Design for doubly reinforced rectangular member undergoing direct stress and bending (example p. 279); assume concrete does not resist tension; take compression steel at twice its elastic value; limit maximum compression in concrete to $0.45f'_c$, stress in compression steel to its tension value, and stress in tensile steel to f_s. (ACI 1109d)

e = eccentricity of resultant load measured from centroid.
F_a = nominal allowable axial unit stress $(0.225f'_c + f_s p_g)$ for spiral columns and 0.8 of this value for tied columns.
F_b = allowable bending unit stress that would be permitted if bending stress only existed.
f_a = nominal axial unit stress = axial load divided by area of member, A_g.
f_b = bending unit stress (actual) = bending moment divided by section modulus of member.
f_{bx}, f_{by} = bending moment components about x and y principal axes divided by the section modulus of the transformed section relative to the respective axes.

FORMULAS—ULTIMATE STRENGTH METHOD †

Although all safe load tables in this book are based upon the working stress method, this section gives a résumé of the ultimate strength method and charts for its application.

* For applications to specific numerical examples, see pages 276, 297, 335.

† For additional information, see ASCE-ACI Joint Committee Report, Proceedings, ASCE, V. 81, Paper 809, Oct., 1955; ACI Journal, Jan., 1956, Proc. V. 52, pp. 505-524; PCA R/C V. 31, 1955, pp. 3-8; Proc., ASCE, V. 82, Paper ST4, July, 1956, and, in particular, Guide for Ultimate Strength Design of Reinforced Concrete, Whitney & Cohen, ACI Journal, Nov., 1956.

ULTIMATE STRENGTH DESIGN

The ultimate strength method increases dead, live, wind, earthquake, and similar loads by suitable load factors, U, and proportions members for ultimate capacity in bending, combined bending and axial load, or direct compression. Ultimate strength design applies only to flexure, not to bond or web reinforcement. For ultimate strength design, external moments and forces are still obtained by the theory of elastic frames (ACI 318-56, 600b).

Notations—Ultimate Strength Only. **For working stress nomenclature, see pages 20-21.**

Loads and load factors:—

U = ultimate strength capacity of section—the maximum combination of thrust and moment the member can sustain prior to failure.
B = effect of basic load consisting of dead load plus volume change due to creep, elastic action, shrinkage, and temperature.
L = effect of live load plus impact.
W = effect of wind load.
E = effect of earthquake forces.
K = load factor.
M_u = ultimate resisting moment.
P_b = ultimate (eccentric) load capacity of a column when failure of both tension steel and compression area occur simultaneously.
P_o = ultimate strength of concentrically loaded short column.
P_u = ultimate strength of eccentrically loaded short column.
P'_u = limitation of eccentric load on long member.

Cross-sectional constants:—

A_{sf} = steel area to develop a total compressive force equal to that of overhanging flange in tee sections.
A_{st} = total area of longitudinal reinforcement $= A_s + A'_s$.
D = total diameter of circular section.
D_s = diameter of circle circumscribing the longitudinal reinforcement in circular section.
e = eccentricity of axial load *measured from the centroid of tensile reinforcement.*
e' = eccentricity of axial load measured from plastic centroid of section.
f_y = yield point of reinforcement, not to be taken greater than 60,000 psi.
k_u = defined by $k_u d$ = distance from extreme compressive fiber to neutral axis at ultimate strength.
k_1 = ratio of average compressive stress to $0.85f'_c$.
k_2 = ratio of distance between extreme compressive fiber and resultant of compressive stresses to distance between extreme fiber and neutral axis.

$m = f_y/0.85f'_c$. $\qquad p_t = A_{st}/A_g$.
$m' = m - 1$. $\qquad p_w = A_s/b'd$.
$p_f = A_{sf}/b'd$. $\qquad q = pf_y/f'_c$.

Load Factors. For structures in which, due to location or proportions, the effects of wind and earthquake loading can be properly neglected:—

$$\left.\begin{aligned} U &= 1.2B + 2.4L \\ U &= K(B + L) \end{aligned}\right\} \text{Use greater.}$$

For structures in which wind loading must be considered:—

$$\left.\begin{aligned} U &= 1.2B + 2.4L + 0.6W \\ U &= 1.2B + 0.6L + 2.4W \\ U &= K(B + L + W/2) \\ U &= K(B + L/2 + W) \end{aligned}\right\} \text{Use greater.}$$

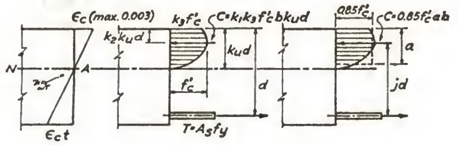
For those structures in which earthquake loading must be considered, substitute E for W in these equations.

The load factor, K, shall be taken equal to 2 for columns and members subjected to combined bending and axial load, and equal to 1.8 for beams and girders subject to bending only.

ULTIMATE STRENGTH DESIGN

General Theory. Ultimate strength design recognizes that strain varies as the distance from the neutral axis and that compressive stress therefore varies as the stress-strain curve of concrete in compression (see Fig.). For computation purposes this may be taken as a rectangle which, at ultimate capacity, has a mean stress intensity of $0.85f'_c$, and a depth a to make the area of the assumed rectangle equal to that of the actual stress-strain curve. Hence the bottom of the assumed stress prism does not coincide with the neutral axis of the member.

1. Rectangular Beams—Tensile Reinforcement Only (Under-reinforced).



$$\Sigma H = 0, \quad C = T, \quad 0.85f_c'ba = A_s f_y = pbdf_y$$

$$\frac{a}{d} = \frac{pf_y}{0.85f'_c} = pm \qquad a = \frac{A_s f_y}{0.85bf'_c} = \frac{A_s m}{b}$$

$$jd = d\left(1 - \frac{a}{2}\right) = d\left(1 - \frac{pf_y}{1.70f'_c}\right) = d\left(1 - \frac{0.59pf_y}{f'_c}\right)$$

$$M_u = Cjd = pbd^2 f_y\left(1 - \frac{0.59pf_y}{f'_c}\right)$$

$M_u = Cjd = bd^2 f'_c\, q(1 - 0.59q) = Tjd = A_s f_y d\,(1 - 0.59p\, f_y/f'_c)$.
for $f'_c \leqq 5000$ psi, max. $p = 0.40f'_c/f_y$
for $f'_c > 5000$ psi, max. $p = [0.40 - 0.000025(f'_c - 5000)]f'_c/f_y$

$$M_u/bd^2 = pf_y\,(1 - 0.59p\,f_y/f'_c) \qquad a/d = \left[1 - \sqrt{1 - \frac{2.35M_u}{f'_c bd^2}}\right]$$

$$p = 1/m - \sqrt{1/m^2 - \frac{2M_u}{f_y m bd^2}}$$

Example. Check the ultimate bending moment in a 12 x 24 in. beam ($d = 21.38$) reinforced with 2-#9 and 1-#11 tension bars only, using ASTM hard grade bars with $f_y = 50{,}000$ psi. This is the beam designed by the working stress method on page 219.

From page 219, $A_s = 3.56$, $p = 0.01387$. From the diagram on page 31-C, with $p = 0.01387$, going across to $f_y = 50{,}000$ and down to $f'_c = 3000$, and across to left, read $M_u/bd^2 = 600$. (This can be computed as $M_u/bd^2 = pf_y\,(1 - 0.59p\,f_y/f'_c) = 0.01387 \times 50{,}000\,(1 - 0.59 \times 0.01387 \times 50{,}000/3000) = 597$). Then $M_u = 597 \times 12 \times 21.38^2 = 3{,}270{,}000$ lb-in. This beam is well below the upper limitation (insuring tension failure) of $p = 0.40f'_c/f_y = 0.40 \times 3000/50{,}000 = 0.024$, but is above the lower limitation $p = 0.18f'_c/f_y = \dfrac{0.18 \times 3000}{50{,}000} = 0.0108$, showing that deflection and the effect of creep must be checked (ACI 318-56 A602e). (Pages 51, 52)

Note that with $f_y = 50{,}000$ psi the ultimate moment M_u is 3,270,000/1,316,000, or 2.48 times the moment by the working stress method on page 219.

This chart works equally well in reverse—entering with a computed value of M_u/bd^2 to find the required p, and then determining the steel area from this.

ULTIMATE STRENGTH DESIGN

2. Rectangular Beams—Compressive Reinforcement (Under-reinforced).

Doubly reinforced beams will not often be required for strength but compressive reinforcement aids greatly in reducing deflection and the effect of creep.

$$M_u = (A_s - A'_s)f_y d\left[\frac{1 - 0.59\,(p - p')\,f_y}{f'_c}\right] + A'_s f_y\,(d - d')$$

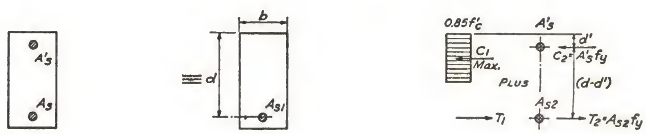
$(p - p')$ is not to exceed values given for p in Section 1.

Example. Check the ultimate bending moment in a doubly-reinforced beam 12 x 24 (d = 21.82 and d' = 2.54) reinforced with 2-#11 and 1-#9 tension bars and 2-#9 compression bars, using ASTM hard grade bars with f_y = 50,000 psi. This is the beam designed by the working stress method on pages 220 and 221.

Compute $p = 4.12/(12 \times 21.82) = 0.0157$ for this beam and compare it with the limitation of $p = 0.40 \times f'_c/f_y = 0.40 \times 3000/50{,}000 = 0.024$ for approximately balanced reinforcement showing that this is to be treated as a rectangular beam reinforced only for tension. However, p exceeds $0.18f'_c/f_y = 0.18 \times 3000/50{,}000 = 0.0108$ by a considerable amount so deflection and the effect of creep must be checked (ACI 318-56 A602c), (see pages 51, 52), and compressive reinforcement might still be desirable.

Example. Since, by ultimate strength design, this beam might not require compressive reinforcement, determine the areas of tension and compression steel if the ultimate bending moment is increased to 6,500,000 lb-in.

Use the same chart as for a rectangular beam with tension reinforcement. Separate the doubly-reinforced beam into two parts, one being a singly-reinforced beam with balanced reinforcement (see Fig.) and the other an internal couple of tension and compression steel with arm $(d - d')$ and area of steel sufficient to make up the difference between the required moment and the moment obtained from balanced reinforcement.



$A_s = A_{s1} + A_{s2}$	$M_1 = 0.306 f'_c bd^2$	$M_2 = M_u - M_1$
$M_u = M_1 + M_2$	$p_1 = 0.40 f'_c/f_y$	$T_2 = C_2 = M_2/(d - d')$
	$A_{s1} = p_1 bd$	$A_{s2} = A'_s = T_2/f_y$

Total Section = Balanced Reinforcement + Compressive Couple

For balanced reinforcement with f_y = 50,000, p = 0.024 (see previous example), for which M_u/bd^2 = 916.

Total moment		M_u = **6,500,000 *lb-in.***
Balanced reinforcement	$M_1 = 916 \times 12 \times 21.82^2$	= **5,250,000**
Left for steel couple		M_2 = **1,250,000 *lb in.***

Since $(d - d') = 21.82 - 2.54 = 19.28$, $C_s = T_2 = 1{,}250{,}000/19.28 = 64{,}600$ lb, and $A_s' = A_{s2} = 64{,}600/50{,}000 = 1.23$ sq in. $A'_s = 1.23$ sq in. requires, say, 1-#7 and 1-#8 = 1.38 sq in. Since $A_s = A_{s1} + A_{s2}$, and A_{s1}, for balanced reinforcement = $0.024 \times 12 \times 21.82 = 6.28$ sq in., $A_s = 6.28 + 1.23 = 7.51$ sq in., say 4-#11 and 1-#10. This is obviously too much steel to be accommodated in a single layer in a 12 in. beam width. Since ultimate strength design takes care of flexure only and the final design must consider shear, bond, and probable deflection as well as the placement of bars

ULTIMATE STRENGTH DESIGN

(items which are illustrated elsewhere), this example has been carried only far enough to illustrate the method. Since $p - p' = 0.024$, which far exceeds $0.18f'_c/f_y$, a check must be made on deflection and the effects of creep (ACI 318-56, A602e), (pages 51, 52).

3. Tee Sections.

When $t \geqq 1.18qd = (1.18p\,f_y/f'_c)d$, design as a rectangular beam, as described in Section 1 above.

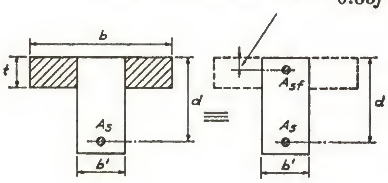
When $t < (1.18p\,f_y/f'_c)d$:—

$$M_u = (A_s - A_{sf})f_y d\,[1 - 0.59\,(p_w - p_f)\,f_y/f'_c] + A_{sf}f_y\,(d - 0.5t)$$

($p_w - p_f$) is not to exceed values given for p in Section 1.

Tee beams are not so often required with ultimate strength design, but when used, the effective flange is limited to an overhang of $6t$ on each side of the web, and the effect of the flanges is evaluated as equivalent to compressive reinforcement, $A_{sf} = 0.85\,f'_c\,(b - b')t/f_y$, located at mid-depth of the compression block. (See Fig.)

0.5t or 0.5a, where $a = \dfrac{A_{s1}f_y}{0.85f'_c b}$ $\qquad A_{sf} = \dfrac{(b - b')t\,0.85f'_c}{f_y}$



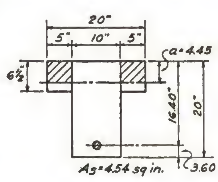
A_{s1} = balanced reinforcement, stem only.

Then design as doubly-reinforced beam.

Example. Check the ultimate moment of the 10 x 20 in. tee beam reinforced with 4.54 sq in. of steel as shown on page 211, using ASTM hard grade bars with $f_y = 50{,}000$ psi.

For the beam stem, compute $p_w = 4.54/(10 \times 16.40) = 0.0276$, which is more than the limiting value found from $p = 0.40f'_c/f_y = 0.40 \times 3000/50{,}000 = 0.024$, so the stem alone is somewhat over-reinforced (so far as ultimate strength is concerned), and additional compression area is required to keep the stress $\leqq 0.85f'_c$.

Using the flange width of 20 in., compute the distance from the top of the beam to the bottom of the stress block (see Fig.)



as $a = \dfrac{A_s f_y}{0.85f'_c b} = \dfrac{4.54 \times 50{,}000}{0.85 \times 3000 \times 20} = 4.45$ in. Thus the bottom of the stress block is well within the $6\frac{1}{2}$ in. slab thickness, so this is to be designed as a rectangular beam:—

$$M_u = 50{,}000 \times 4.54 \times \left(16.40 - \frac{4.45}{2}\right) = 3{,}210{,}000 \text{ lb-in.}$$

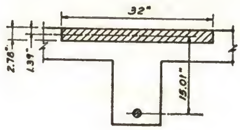
or:—

$$M_u = 0.85 \times 3000 \times 20 \times 4.45\left(16.40 - \frac{4.45}{2}\right) = 3{,}210{,}000 \text{ lb-in.}$$

Example. If the available width of flange in the previous problem had been 32 in., determine the minimum flange thickness and ultimate resisting moment.

It is permissible to use in calculations at a uniform compressive stress of $0.85f'_c$ as much of the slab thickness as is required to resist the pull of the tension steel. This somewhat increases the arm of the internal couple and somewhat increases the ultimate moment without affecting the actual disposition of compression in the concrete, the rectangular stress block being only a convenient approximation at best.

ULTIMATE STRENGTH DESIGN



The total tension can then be computed as $4.54 \times 50{,}000 = 227{,}000$ lb, which must also be the total compression distributed at $0.85 \times 3000 = 2550$ psi over a flange width of 32 in., whose depth can be determined as $\dfrac{227{,}000}{2550 \times 32} = 2.78$ in. This makes the arm of the internal couple $16.40 - \dfrac{2.78}{2} = 15.01$ in. and the resisting moment:

$$M_u = 227{,}000 \times 15.01 = 3{,}420{,}000 \text{ lb in.}$$

which is 6½ per cent greater than in the previous example.

The ultimate capacity of this beam is approximately $\dfrac{3{,}210{,}000}{1{,}289{,}000} = 2\frac{1}{2}$ times that obtained on page 219 by the working stress method.

4. Concentrically Loaded Short Columns.

ACI 318-56 requires us to design all members sustaining direct axial load for a minimum eccentricity of $0.05t$ for spirally reinforced columns, $0.10t$ for tied columns, by the methods given in Section 5.

To facilitate the direct design of compression members with loads that can be assumed as reasonably concentric, two charts are offered (one for tied columns and one for spirally reinforced) in which the minimum eccentricities have been incorporated into the methods of Section 5, and the required values can then be read directly.

Example. By the ultimate strength method determine the ultimate carrying capacity of a tied column 35 in. square of 3000 psi concrete, reinforced with 18-#10 bars of ASTM intermediate grade steel, $f_y = 40{,}000$ psi, which is the example solved by the working stress method on page 235.

Compute $p_t = \dfrac{18 \times 1.27}{35 \times 35} = 0.01865$, $d/t = 32.5/35 = 0.93$, and $p_t m' = 0.01865\left(\dfrac{40{,}000}{0.85 \times 3{,}000} - 1\right) = 0.274$. Then from the chart on page 31-D read $K = 0.835$, from which $P_u = Kbtf'_c = 0.835 \times 35 \times 35 \times 3000 = 3{,}090{,}000$ lb.

This ultimate capacity is $3{,}090{,}000/954{,}100 = 3.24$ times the safe working capacity determined on page 235.

Example. By the ultimate strength method determine the ultimate carrying capacity of a spirally reinforced round column, 40 in. diameter, reinforced with 28-#11 vertical bars of ASTM intermediate grade steel, $f_y = 40{,}000$ psi, which is the example solved by the working stress method on page 250.

Compute $p_t = \dfrac{28 \times 1.56}{\pi \times 20 \times 20} = 0.0348$, $d/D = 37.5/40 = 0.94$, and $p_t m' = 0.0348\left(\dfrac{40{,}000}{0.85 \times 3000} - 1\right) = 0.511$. Then from the chart on page 31-D read $K = 0.845$ and compute $P_u = KD^2f'_c = 0.845 \times 40 \times 40 \times 3{,}000 = 4{,}050{,}000$ lb.

This ultimate capacity is $4{,}050{,}000/1{,}547{,}100 = 2.62$ times the safe working capacity determined on page 250.

5. Bending and Axial Load.

A. Rectangular Section:—

$$\boldsymbol{P_u = 0.85f'_c bdk_u k_1 + A'_s f_y - A_s f_s}$$
$$\boldsymbol{P_u e = 0.85f'_c bd^2 k_u k_1 (1 - k_2 k_u) + A'_s f_y (d - d')}$$

Limitations:—

$k_u < 1$. $k_2/k_1 \ngtr 0.5$.
$k_1 \nless 0.85$ when $f'_c \nless 5000$
$k_1 \nless 0.85 - 0.00005(f'_c - 5000)$ when $f'_c > 5000$.

ULTIMATE STRENGTH DESIGN

Probable failure in tension when $P_u \lessgtr P_b = 0.85k_1\left(\dfrac{90{,}000}{90{,}000 + f_y}\right) f'_c bd + A'_s f_y - A_s f_y$, and failure in compression when $P_u > P_b$.

Controlled by tension:—

$$P_u = 0.85f'_c bd\, p'm' - pm + (1 - e/d) + \sqrt{(1 - e/d)^2 + 2\,[(e/d)\,(pm - p'm') + p'm'\,(1 - d'/d)]}$$

Controlled by compression:

$$P_u = \frac{A'_s f_y}{\dfrac{e'}{(d - d')} + \dfrac{1}{2}} + \frac{btf'_c}{\dfrac{3te'}{d^2} + 1.18}$$

Four charts are presented for eccentrically loaded compression members for values of d/t = 0.80, 0.85, 0.90, and 0.95. They apply equally well whether tension or compression controls, but in the latter case, enter with $p_t m'$ instead of $p_t m$.

Example. By the ultimate strength method, determine the amount of reinforcement, using ASTM hard grade bars with f_y = 50,000 psi, required in a column 20 x 20 in., carrying a load of 400 kips, with an eccentricity of 7 in. This corresponds roughly with the column designed by the working stress method on page 279.

P_u = 400,000; d = 17.5; d' = 2.5; d/t = 17.5/20 = 0.875, interpolate between charts on pages 31E-31F for d/t = 0.85 and 0.90.

Enter on left side with $P_u/f'_c bd = \dfrac{400{,}000}{3000 \times 20 \times 20} = 0.333$, and on the bottom with $P_u e/f'_c bd^2 = \dfrac{400{,}000 \times 7}{3000 \times 20 \times 20^2} = 0.117$, then estimate from curved lines $p_t m = \dfrac{0.007 + 0.010}{2} = 0.085$.

$m = f_y/0.85f'_c = 50{,}000/0.85 \times 3000 = 19.6.$

$m' = m - 1 = 19.6 - 1 = 18.6.$

$p_t = 0.085/18.6 = 0.0046$ $A_s = 0.0046 \times 20 \times 20 = 1.84$ sq in., but ACI 1104a requires a minimum of $0.01 \times 20 \times 20 = 4.00$ sq in.

B. Circular Section:—

controlled by tension:—

$$P_u = 0.85f'_c D^2 \left[\sqrt{(0.85e'/D - 0.38)^2 + p_t m D_s/2.5D} - (0.85e'/D - 0.38)\right]$$

controlled by compression:—

$$P_u = \frac{A_{st} f_y}{\dfrac{3e'}{D_s} + 1} + \frac{A_g f'_c}{\dfrac{9.6De'}{(0.8D + 0.67D_s)^2} + 1.18}$$

C. Long Members:— $(H/t > 15)$

$$P'_u = P_o\,(1.6 - 0.04\,H/t)$$

Charts are available * for solving circular members undergoing combined bending and direct stress as well as other problems in ultimate strength design. Those presented here will be found sufficient for the majority of problems.

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* Obtainable from American Concrete Institute, 18263 West McNichols Road, Detroit 19, Michigan.

ULTIMATE STRENGTH DESIGN

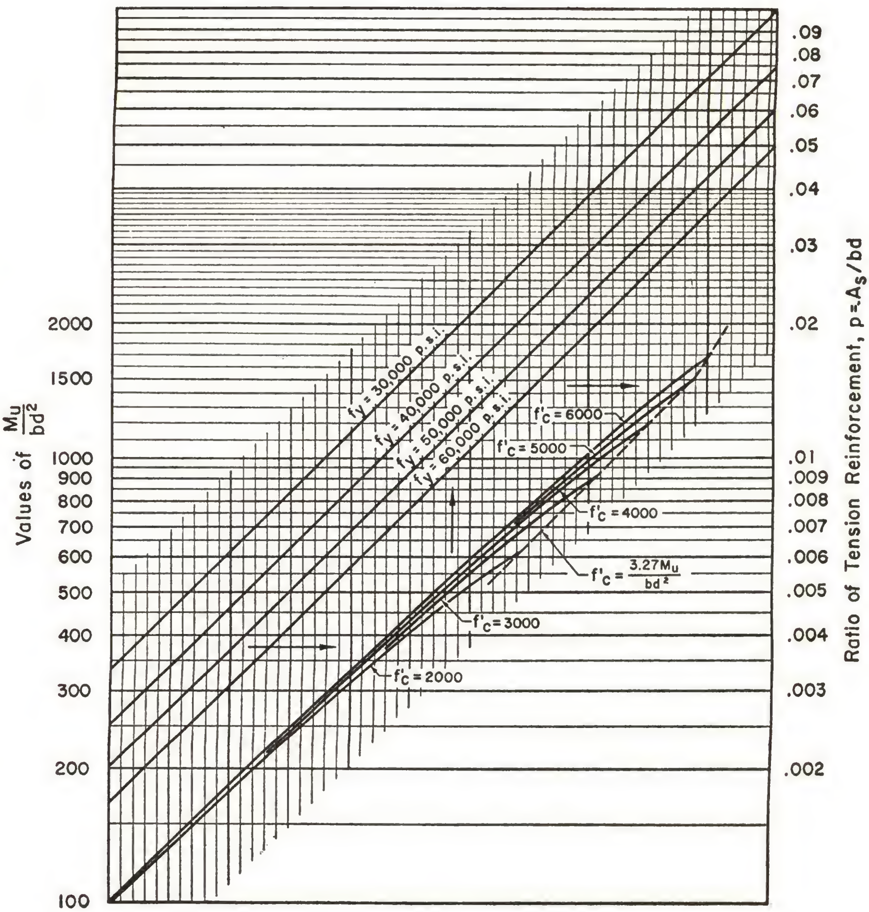


CHART I

MOMENT CAPACITY OF RECTANGULAR SECTIONS WITHOUT COMPRESSION REINFORCEMENT

ULTIMATE STRENGTH DESIGN

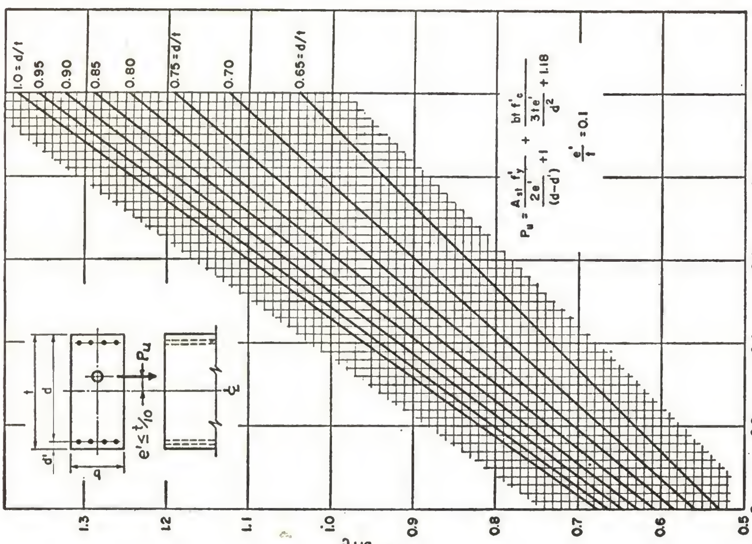


CHART II

DIRECT LOAD CAPACITY OF RECTANGULAR SECTIONS WITH SYMMETRICAL REINFORCEMENT FOR ECCENTRICITIES LESS THAN OR EQUAL TO $\frac{t}{10}$

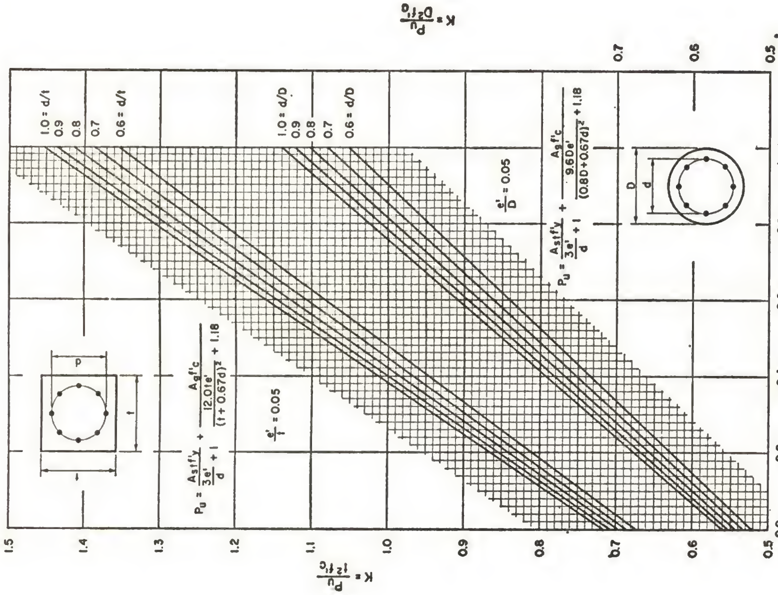
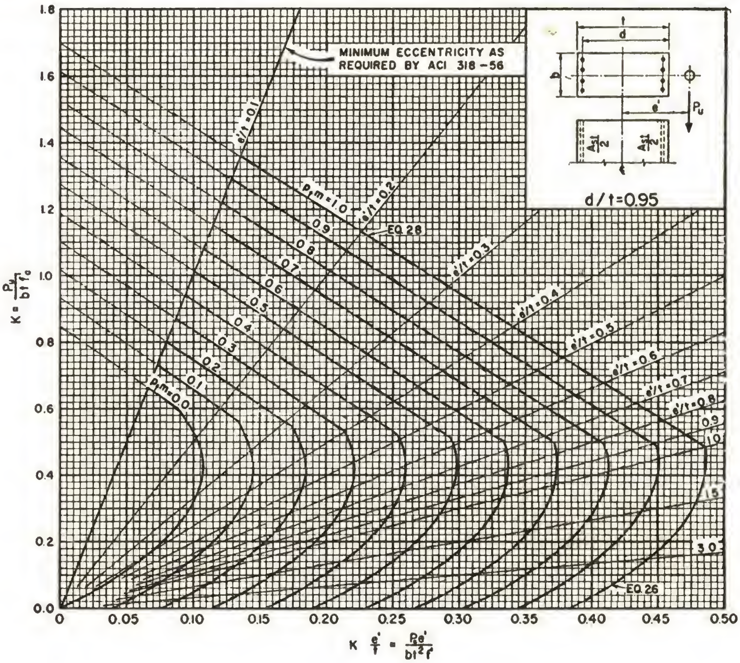


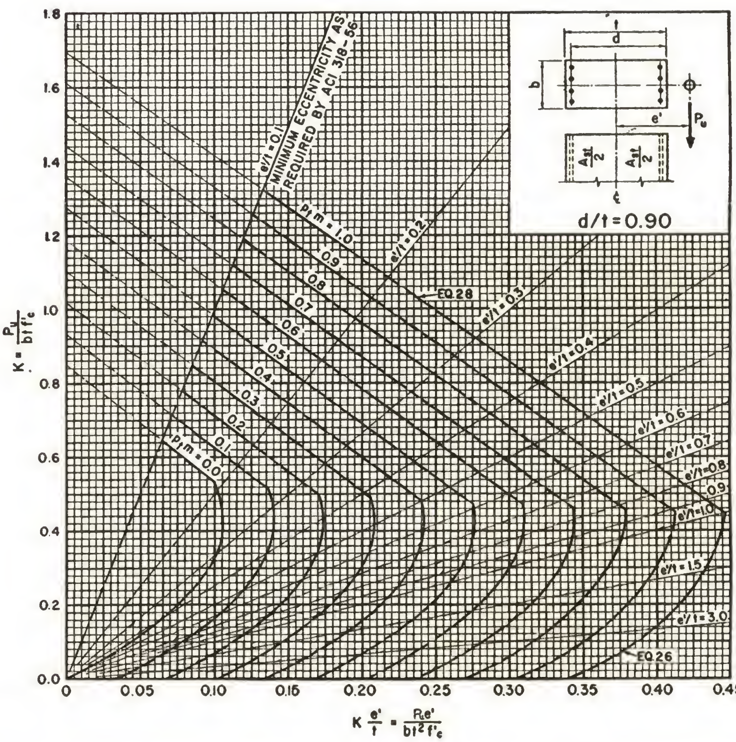
CHART III

DIRECT LOAD CAPACITY OF SQUARE SECTIONS WITH SPIRAL REINFORCEMENT AND CIRCULAR SECTIONS WITH SPIRAL REINFORCEMENT FOR ECCENTRICITIES LESS THAN OR EQUAL TO $\frac{t}{20}$

ULTIMATE STRENGTH DESIGN

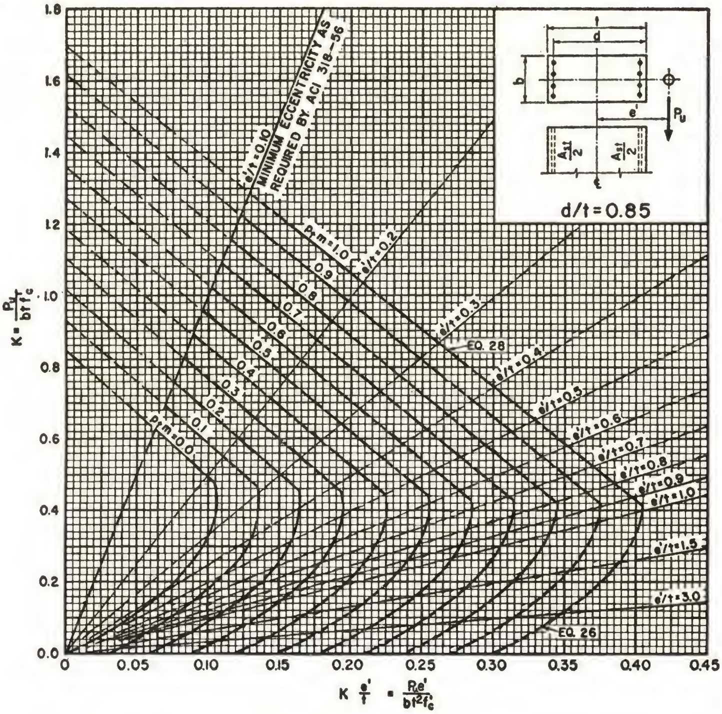
Bending and axial load—d/t = 0.90 or 0.95, rectangular sections with symmetrical reinforcement.

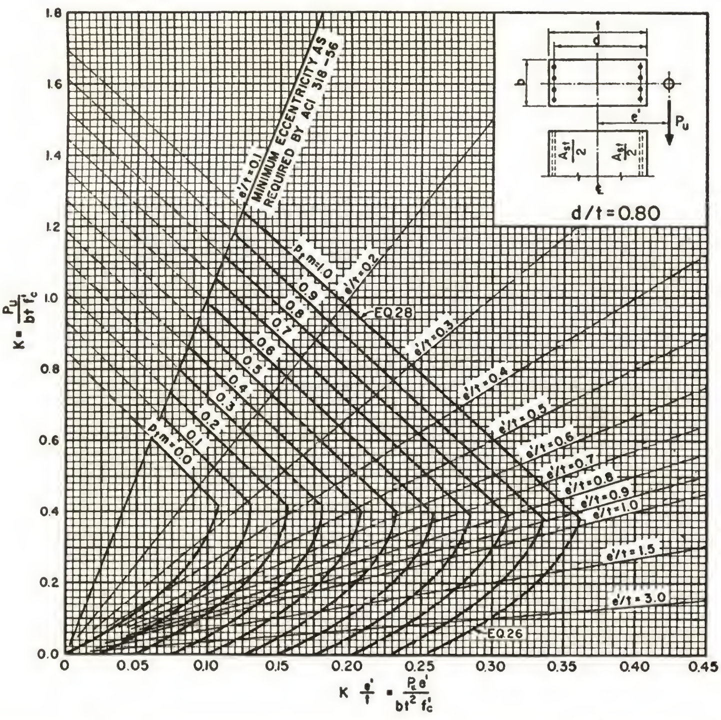




ULTIMATE STRENGTH DESIGN

Bending and axial load—d/t = 0.80 or 0.85, rectangular sections with symmetrical reinforcement.





ALLOWABLE UNIT WORKING STRESSES

From "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

The table gives the allowable working stresses for various grades of concrete with bars whose deformations meet ASTM A305 and for those which do not. The stresses used throughout this book for bond and diagonal tension in computing safe load tables apply only to bars with deformations conforming to ASTM A305.

CONCRETE STRESSES

Description	For Any Strength of Concrete $n = \frac{30,000}{f'_c}$	Maximum Value psi	For Strength of Concrete Shown Below f'_c 2000 $n = 15$	2500 $n = 12$	3000 $n = 10$	3750 $n = 8$	5000 $n = 6$
Flexure: f_c							
Extreme fiber stress in compression	$0.45f'_c$		900	1125	1350	1688	2250
Extreme fiber stress tension plain concrete footings	$0.03f'_c$		60	75	90	113	150
Shear: v (as a measure of diagonal tension)							
Beams with no web reinforcement	$0.03f'_c$	90	60	75	90	90	90
Beams with longitudinal bars and with either stirrups or properly located bent bars	$0.08f'_c$	240	160	200	240	240	240
Beams with longitudinal bars and a combination of stirrups and bent bars (the latter bent up suitably to carry at least $0.04f'_c$)	$0.12f'_c$	360	240	300	360	360	360
Footings	$0.03f'_c$	75	60	75	75	75	75
(Flat slabs, ACI 318-56 Ch. 10)							
Bond: u							
Deformed bars							
Top bars *	$0.07f'_c$	245	140	175	210	245	245
In 2-way footings (except top bars)	$0.08f'_c$	280	160	200	240	280	280
All others	$0.10f'_c$	350	200	250	300	350	350
Plain bars (must be hooked)							
Top bars*	$0.03f'_c$	105	60	75	90	105	105
In 2-way footings (except top bars)	$0.036f'_c$	126	72	90	108	126	126
All others	$0.045f'_c$	158	90	113	135	158	158
Bearing: f_c							
On full area	$0.25f'_c$		500	625	750	938	1250
On one-third area or less †	$0.375f'_c$		750	938	1125	1405	1875

* Bars near top of beams and girders having more than 12 in. of concrete under bars.
† Increase permitted only when least distance between edges of loaded and unloaded areas is a minimum of one-fourth parallel side of loaded area. Allowable bearing on reasonably concentric area greater than one-third but less than full area shall be interpolated.

STEEL STRESSES

Tension (f_s): 20,000 psi on rail steel, intermediate and hard grades of billet steel or of axle steel, and cold-drawn steel wire.
18,000 psi for structural grade billet or axle steel.

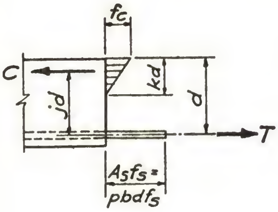
Web reinforcement (f_v): same as f_s

Tension (one way slabs, spans $\not>$ 12 ft) Bars $\frac{3}{8}\phi$ or less: 50% yield point but $\not>$ 30,000 psi.

Compression (column verticals) (f_s): 40% yield point but $\not>$ 30,000 psi.; (metal cores) (f_r): steel sections 16,000 psi; cast iron sections 10,000 psi; steel pipe—ACI 318–56 (1106b)

VALUE OF *n, p, k, j, R* FOR VARIOUS COMBINATIONS OF STEEL AND CONCRETE STRESSES FOR RECTANGULAR BEAMS AND SLABS

(WORKING STRESS METHOD)



$n = \dfrac{30,000}{f'_c}$ $\qquad k = \dfrac{1}{1 + \dfrac{f_s}{nf_c}}$ $\qquad p = \dfrac{f_c k}{2f_s}$ $\qquad j = 1 - \dfrac{k}{3}$

$R = \tfrac{1}{2} f_c k j = p f_s j \qquad f_c = 0.45\, f'_c$

f'_c	1500	2000	2500	3000	3750	5000
f_c	675	900	1125	1350	1687	2250
n	20	15	12	10	8	6
			$f_s = 14,000$			
p	.01183	.01578	.01971	.02363	.02954	.03942
k	.4909	.4909	.4909	.4909	.4909	.4909
j	.8364	.8364	.8364	.8364	.8364	.8364
R	138.5	184.8	230.8	277.0	346.2	461.8
			$f_s = 16,000$			
p	.00965	.01288	.01609	.01930	.02412	.03218
k	.4576	.4576	.4576	.4576	.4576	.4576
j	.8475	.8475	.8475	.8475	.8475	.8475
R	130.8	174.6	218.1	261.6	326.8	436.0
			$f_s = 18,000$			
p	.008035	.01071	.01339	.01605	.02005	.02678
k	.4286	.4286	.4286	.4286	.4286	.4286
j	.8571	.8571	.8571	.8571	.8571	.8571
R	124.0	165.2	206.4	247.8	309.2	412.8
			$f_s = 20,000$			
p	.006800	.009065	.0113	.01360	.01700	.02267
k	.4030	.4030	.4030	.4030	.4030	.4030
j	.8657	.8657	.8657	.8657	.8657	.8657
R	117.8	157.0	196.0	235.4	294.2	392.6
			$f_s = 24,000$			
p	.005062	.006750	.00843	.01012	.01263	.01688
k	.3600	.3600	.3600	.3600	.3600	.3600
j	.8800	.8800	.8800	.8800	.8800	.8800
R	107.0	142.6	178.0	213.9	266.9	356.1
			$f_s = 30,000$			
p	.003490	.004650	.005822	.006980	.008720	.01164
k	.3103	.3103	.3103	.3103	.3103	.3103
j	.8966	.8966	.8966	.8966	.8966	.8966
R	93.75	125.1	156.5	187.7	234.5	312.9

DESIGN CONSTANTS—ALL PERCENTAGES REINFORCEMENT
20,000 psi STEEL AND 3,000 psi CONCRETE ONLY
(WORKING STRESS METHOD)

$p = \frac{A_s}{bd}$ $\quad k = \sqrt{2pn + (pn)^2} - pn$ $\quad j = 1 - \frac{k}{3}$ $\quad R_s = pf_sj$ $\quad R_c = \frac{1}{2}f_ckj$

p	k	j	R_s	R_c	p	k	j	R_s	R_c
0.0010	0.1317	0.9561	19.1	84.9	0.0114	0.3769	0.8744	199.3	222.4
0.0012	0.1434	0.9522	22.8	92.1	0.0116	0.3794	0.8735	202.6	223.7
0.0014	0.1539	0.9487	26.5	98.5	0.0118	0.3819	0.8727	205.9	224.9
0.0016	0.1636	0.9455	30.2	104.4	0.0120	0.3844	0.8719	209.2	226.2
0.0018	0.1726	0.9425	33.9	109.8	0.0122	0.3868	0.8711	212.5	227.4
0.0020	0.1808	0.9397	37.5	114.6	0.0123	0.3880	0.8707	214.1	228.0
0.0022	0.1889	0.9370	41.2	119.4	0.0124	0.3892	0.8703	215.8	228.6
0.0024	0.1964	0.9345	44.8	123.8	0.0125	0.3904	0.8699	217.5	229.2
0.0026	0.2035	0.9322	48.4	128.0	0.0126	0.3916	0.8695	219.1	229.8
0.0028	0.2103	0.9299	52.0	132.0	0.0127	0.3927	0.8691	220.7	230.3
0.0030	0.2168	0.9277	55.6	135.7	0.0128	0.3939	0.8687	222.3	230.9
0.0032	0.2229	0.9257	59.2	139.2	0.0129	0.3950	0.8683	224.0	231.5
0.0034	0.2290	0.9237	62.8	142.7	0.0130	0.3962	0.8679	225.6	232.1
0.0036	0.2347	0.9218	66.3	146.0	0.0131	0.3973	0.8676	227.3	232.6
0.0038	0.2403	0.9199	69.9	149.2	0.0132	0.3985	0.8672	228.9	233.2
0.0040	0.2457	0.9181	73.4	152.2	0.0133	0.3996	0.8668	230.5	233.8
0.0042	0.2509	0.9164	76.9	155.2	0.0134	0.4007	0.8664	232.2	234.3
0.0044	0.2559	0.9147	80.4	158.0	0.0135	0.4018	0.8661	233.8	234.9
0.0046	0.2608	0.9131	84.0	160.7	**0.0136**	**0.4030**	**0.8657**	**235.4**	**235.4**
0.0048	0.2655	0.9115	87.5	163.3	0.0137	0.4041	0.8653	237.0	236.0
0.0050	0.2701	0.9100	91.0	165.9	0.0138	0.4052	0.8649	238.7	236.5
0.0052	0.2747	0.9084	94.4	168.4	0.0139	0.4062	0.8646	240.3	237.0
0.0054	0.2790	0.9070	97.9	170.8	0.0140	0.4074	0.8642	241.9	237.6
0.0056	0.2833	0.9056	101.4	173.1	0.0141	0.4084	0.8639	243.6	238.1
0.0058	0.2875	0.9042	104.8	175.4	0.0142	0.4095	0.8635	245.2	238.6
0.0060	0.2914	0.9029	108.3	177.6	0.0143	0.4105	0.8632	246.8	239.1
0.0062	0.2956	0.9015	111.7	179.8	0.0144	0.4116	0.8628	248.4	239.7
0.0064	0.2994	0.9002	115.2	181.9	0.0145	0.4127	0.8624	250.1	240.2
0.0066	0.3033	0.8989	118.6	184.0	0.0146	0.4137	0.8621	251.7	240.7
0.0068	0.3070	0.8977	122.0	186.0	0.0147	0.4148	0.8617	253.3	241.2
0.0070	0.3107	0.8964	125.5	187.9	0.0148	0.4158	0.8614	254.9	241.7
0.0072	0.3142	0.8953	128.9	189.8	0.0149	0.4168	0.8611	256.6	242.2
0.0074	0.3178	0.8941	132.3	191.8	0.0150	0.4179	0.8607	258.2	242.7
0.0076	0.3212	0.8929	135.7	193.5	0.0152	0.4199	0.8601	261.4	243.7
0.0078	0.3246	0.8918	139.1	195.4	0.0154	0.4219	0.8594	264.7	244.7
0.0080	0.3279	0.8907	142.5	197.1	0.0156	0.4239	0.8587	267.9	245.7
0.0082	0.3312	0.8896	145.8	198.8	0.0158	0.4259	0.8581	271.1	246.6
0.0084	0.3344	0.8885	149.2	200.5	0.0160	0.4279	0.8574	274.3	247.6
0.0086	0.3376	0.8875	152.6	202.2	0.0162	0.4298	0.8567	277.5	248.5
0.0088	0.3407	0.8864	156.0	203.8	0.0164	0.4317	0.8561	280.8	249.4
0.0090	0.3437	0.8854	159.3	205.4	0.0166	0.4336	0.8555	284.0	250.3
0.0092	0.3467	0.8844	162.7	206.9	0.0168	0.4355	0.8548	287.2	251.2
0.0094	0.3497	0.8834	166.0	208.5	0.0170	0.4374	0.8542	290.4	252.2
0.0096	0.3526	0.8825	169.4	210.0	0.0172	0.4392	0.8536	293.6	253.0
0.0098	0.3554	0.8815	172.7	211.4	0.0174	0.4410	0.8530	296.8	253.9
0.0100	0.3583	0.8806	176.1	212.9	0.0176	0.4428	0.8524	300.0	254.7
0.0102	0.3610	0.8797	179.4	214.3	0.0178	0.4446	0.8518	303.2	255.6
0.0104	0.3638	0.8787	182.7	215.7	0.0180	0.4464	0.8512	306.4	256.4
0.0106	0.3665	0.8778	186.0	217.1	0.0185	0.4508	0.8497	314.3	258.5
0.0108	0.3691	0.8770	189.4	218.5	0.0190	0.4551	0.8483	322.3	260.5
0.0110	0.3718	0.8761	192.7	219.8	0.0195	0.4592	0.8469	330.2	262.5
0.0112	0.3744	0.8752	196.0	221.1	0.0200	0.4633	0.8456	338.2	264.4

VALUES OF $R = \frac{M}{bd^2}$ FOR TEE BEAMS, BALANCED REINFORCEMENT
(WORKING STRESS METHOD)
$f_s = 20{,}000$ psi

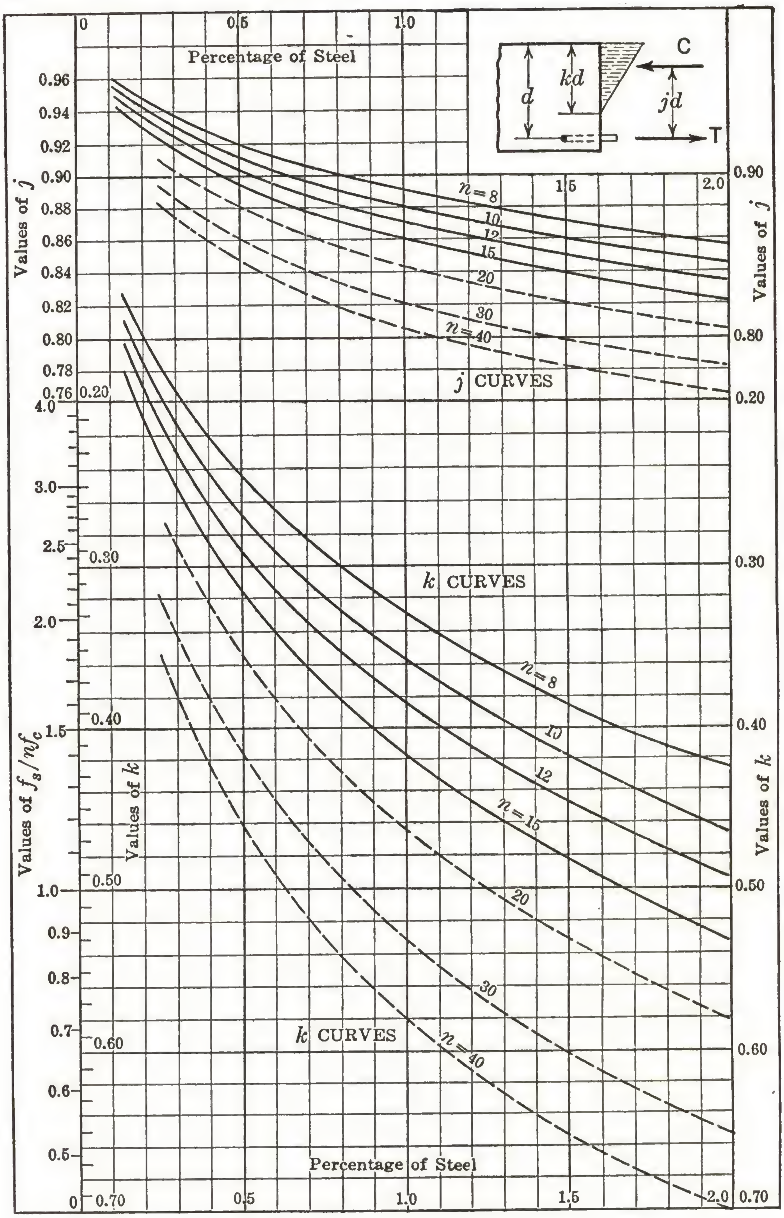
f'_c and n	f_c	t/d 0.06	0.08	0.10	0.12	0.14	0.16	0.18	0.20	0.24	0.28	0.32	0.36	0.40
	500	26	34	40	46	52	57	61	64	69	73	74	*74	*74
	550	29	37	45	52	58	63	68	72	78	83	85	*85	*85
1500	600	32	41	49	57	64	70	75	80	88	93	97	98	*98
	650	35	45	54	62	70	77	83	88	97	104	108	110	*110
	700	38	48	58	68	76	83	90	96	106	114	119	123	124
20	750	40	52	63	73	82	90	98	104	116	124	131	135	137
	800	43	56	67	78	88	97	105	113	125	135	142	147	150
	900	49	63	76	89	100	110	120	128	144	156	165	172	176
	650	35	44	53	60	67	73	78	83	89	93	95	*95	*95
	700	37	48	57	66	73	80	86	91	99	104	106	107	*107
2000	750	40	51	62	71	79	87	93	99	108	114	118	119	*119
	800	43	55	66	76	85	93	101	107	117	125	129	131	*131
	900	49	62	75	87	97	107	116	123	136	145	152	156	157
15	1000	54	70	84	97	110	121	131	140	155	166	175	180	183
	1200	65	84	102	119	134	148	160	172	192	208	221	229	235
	1350	74	96	116	134	152	168	183	196	220	240	255	266	275
	800	43	54	65	74	82	90	96	101	109	114	116	*116	*116
	875	47	60	71	82	91	100	107	113	123	130	133	*133	*133
2500	950	51	65	78	90	100	110	118	126	137	145	150	152	*152
	1000	54	69	83	95	107	117	126	134	147	156	161	164	*164
	1125	61	78	94	108	122	134	145	154	170	181	190	195	196
12	1250	68	87	105	122	137	151	163	174	193	208	219	225	229
	1500	82	106	128	148	167	185	201	215	240	260	276	287	294
	1700	93	120	146	169	191	212	231	247	278	302	321	336	346
	975	52	66	79	90	101	109	117	124	134	140	142	*142	*142
	1050	56	71	86	98	110	120	128	136	148	156	159	160	*160
3000	1125	60	77	92	106	119	130	140	148	162	171	176	178	*178
	1200	65	83	99	114	128	140	151	161	176	187	194	197	*197
	1350	73	94	113	130	146	160	173	185	204	218	228	234	236
10	1500	81	105	126	146	164	181	196	209	232	250	262	271	275
	1800	98	126	153	178	201	222	241	258	288	312	331	344	353
	2025	111	143	174	202	228	252	274	295	330	359	382	399	411
	1200	64	81	97	111	123	134	144	152	164	171	173	*173	*173
	1300	69	88	106	121	136	148	159	168	183	192	196	197	*197
3750	1400	75	96	115	132	148	162	174	184	201	213	219	221	*221
	1500	81	103	124	143	160	175	189	201	220	234	242	246	*246
	1700	92	118	142	164	184	202	219	233	257	276	288	295	298
8	1875	102	131	158	183	205	226	245	262	290	312	328	338	344
	2250	123	158	191	222	251	277	301	323	359	390	414	431	441
	2525	138	179	216	252	284	314	342	367	412	448	477	498	514
	1500	79	101	120	137	151	165	176	186	200	206	208	*208	*208
	1700	90	115	138	158	176	193	206	218	237	249	254	*254	*254
5000	1875	100	128	154	177	197	216	233	247	269	285	294	297	*297
	2250	121	155	187	216	243	267	288	307	339	363	379	388	392
	2525	137	176	212	246	276	304	330	352	390	420	442	456	464
6	2800	152	196	237	275	310	342	371	397	441	477	505	524	536
	3000	163	211	255	296	334	368	400	430	479	520	550	573	588
	3300	180	231	282	328	371	409	445	478	535	581	619	646	665

* The values in boldface type indicate that the neutral axis is within the flange.

Acknowledgments:—Pages 36 to 44, inclusive, are reproduced by permission from Turneaure and Mauer "Principles of Reinforced Concrete Construction," John Wiley and Sons, Inc., 1935; Pages 45, 46, 47, from Sutherland and Reese "Reinforced Concrete Design," John Wiley and Sons, Inc., 1943; Page 48 is adapted from A. R. Lord "Handbook of Reinforced Concrete Building Design" of the ACI, 1928.

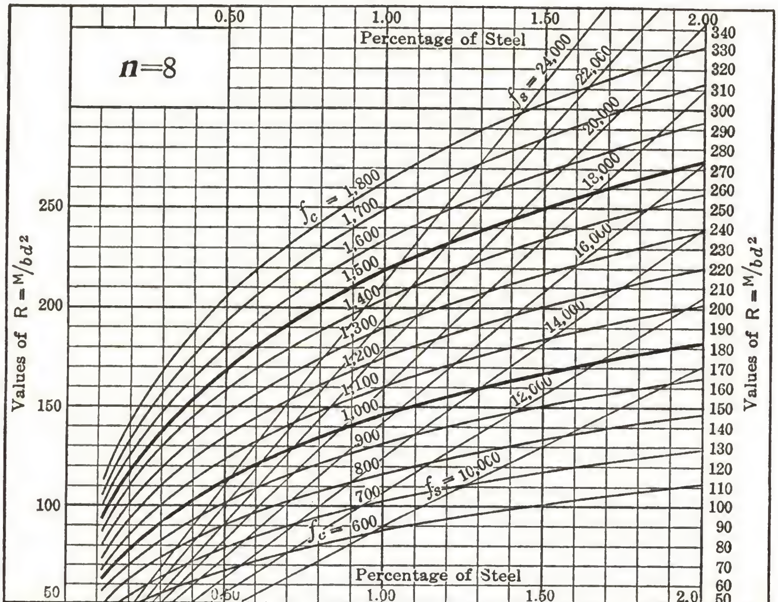
VALUES OF k AND j FOR RECTANGULAR BEAMS.

(WORKING STRESS METHOD)

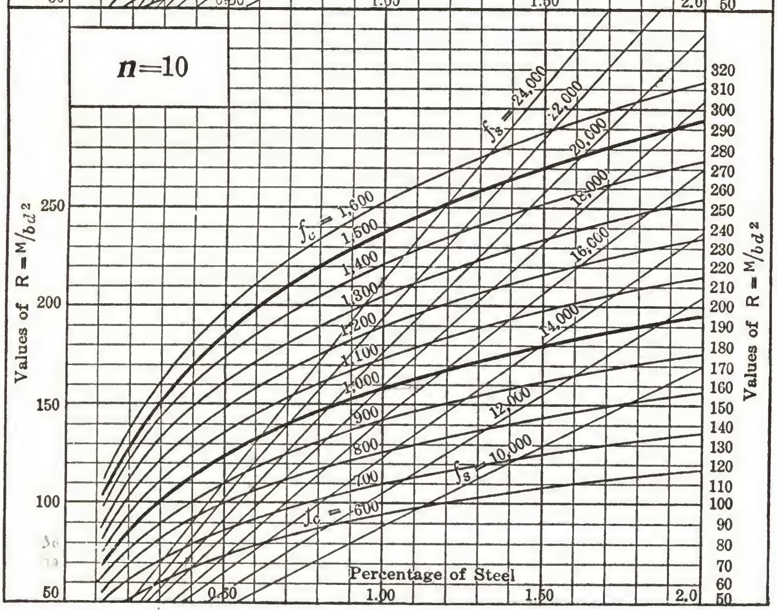


COEFFICIENTS OF RESISTANCE OF RECTANGULAR BEAMS. $M = Rbd^2$.

(WORKING STRESS METHOD)



3750



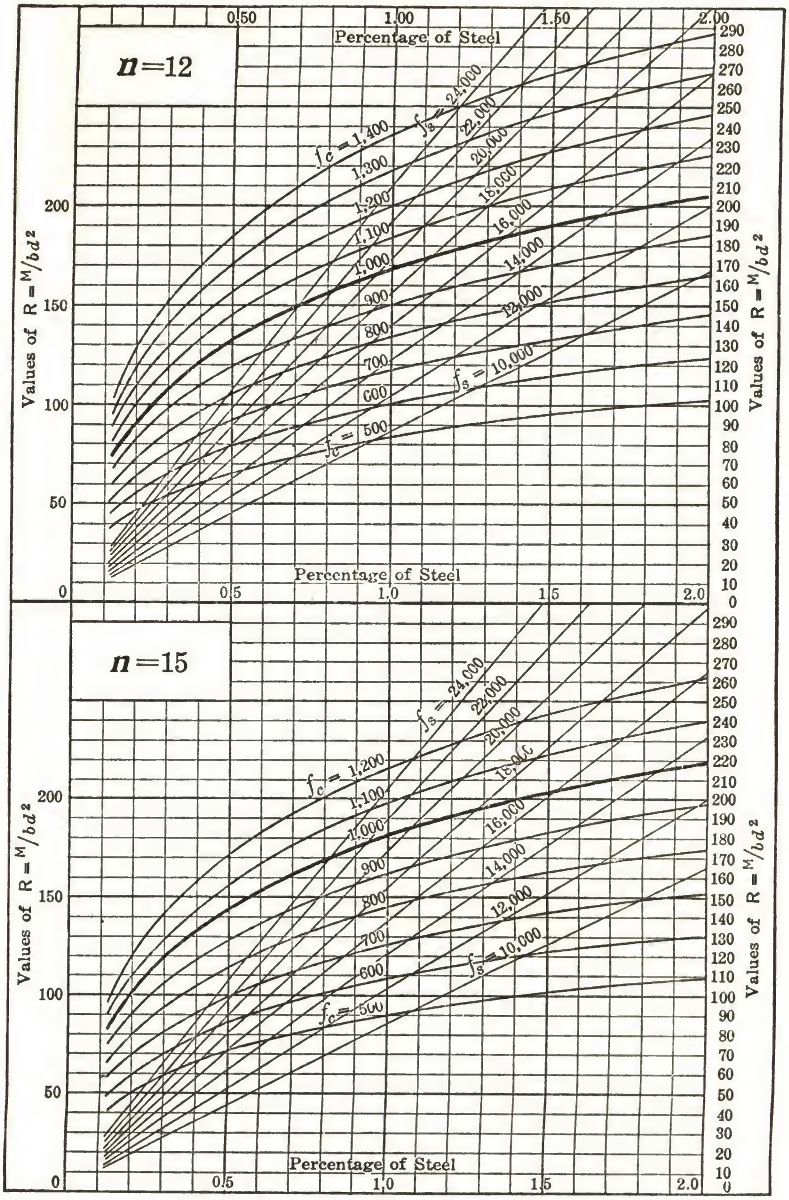
3000

COEFFICIENTS OF RESISTANCE OF RECTANGULAR BEAMS. $M = Rbd^2$.

(WORKING STRESS METHOD)

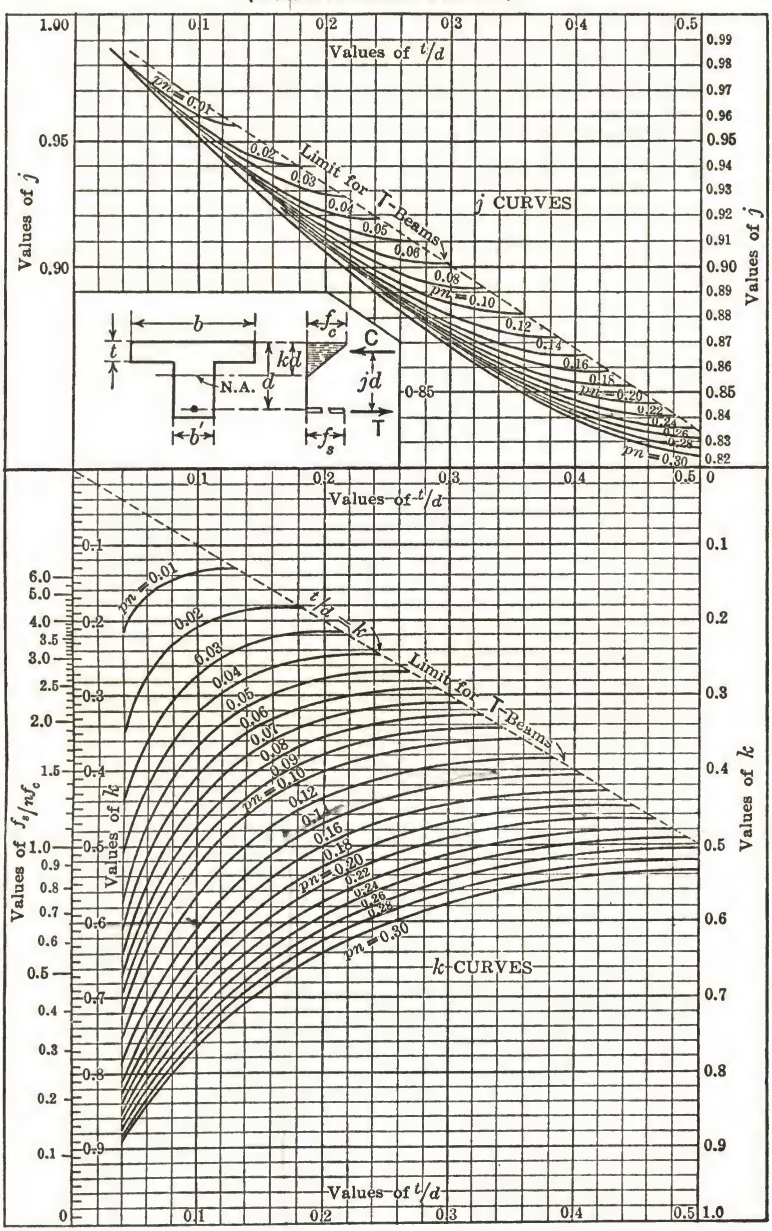
2500

2000



VALUES OF k AND j FOR TEE-BEAMS

(WORKING STRESS METHOD)

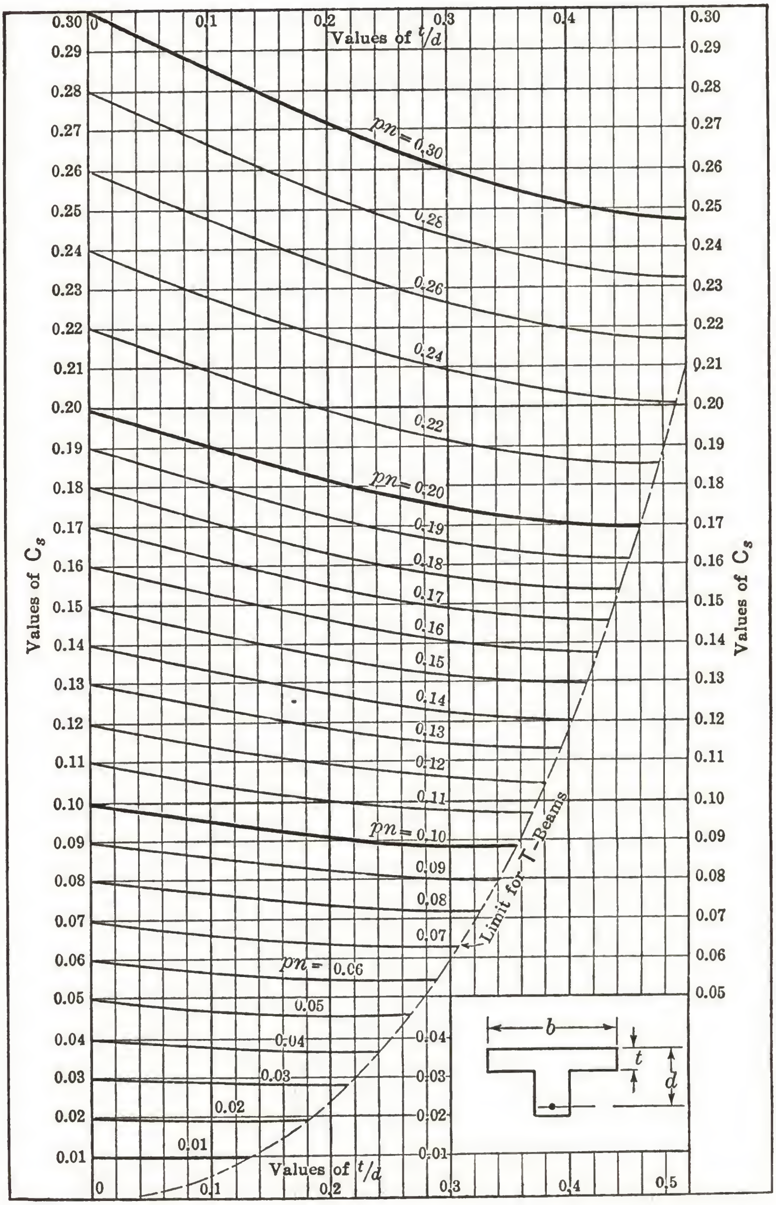


For $f_s = 20{,}000$, $f_c = 1350$, $n = 10$, $\frac{f_s}{nf_c} = 1.48$.

COEFFICIENTS OF RESISTANCE OF TEE-BEAMS WITH RESPECT TO STEEL.

(WORKING STRESS METHOD)

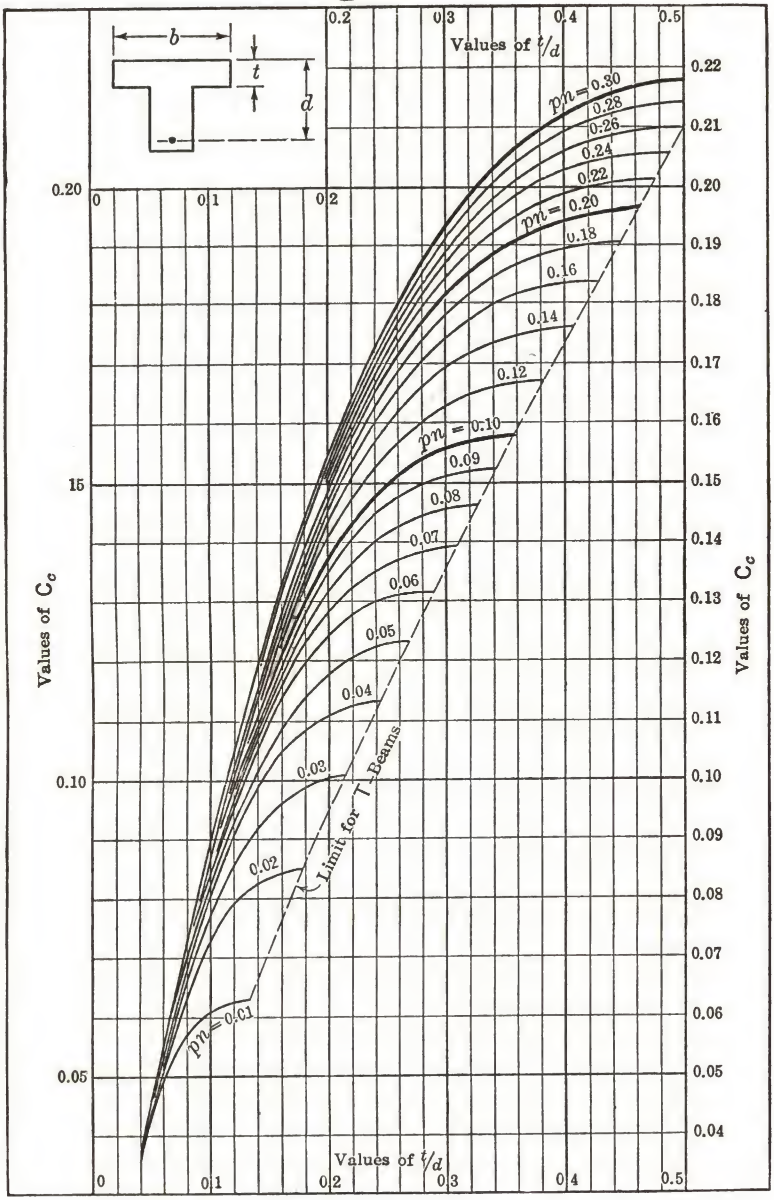
$$M_s = C_s \frac{f_s}{n} bd^2.$$



COEFFICIENTS OF RESISTANCE OF TEE-BEAMS WITH RESPECT TO CONCRETE.

(WORKING STRESS METHOD)

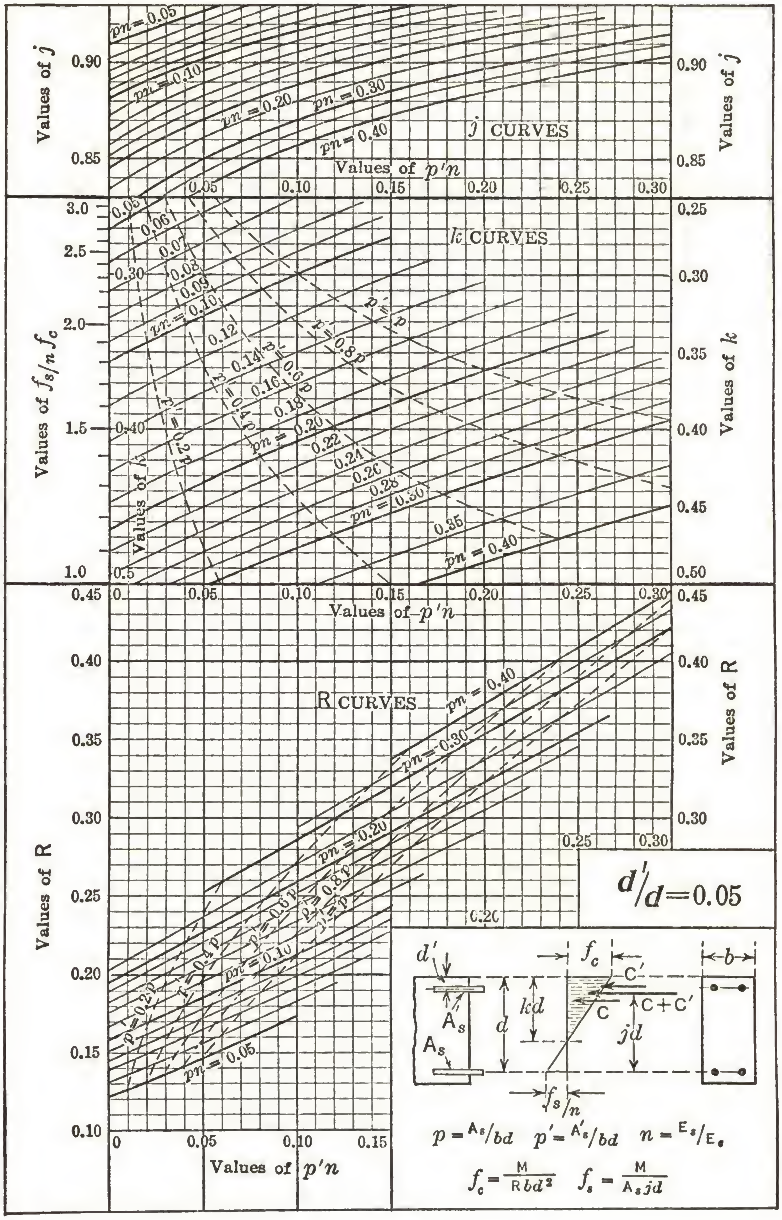
$M_c = C_c f_c b d^2.$



RECTANGULAR BEAMS REINFORCED FOR COMPRESSION. $M = f_c R b d^2$.

(WORKING STRESS METHOD)

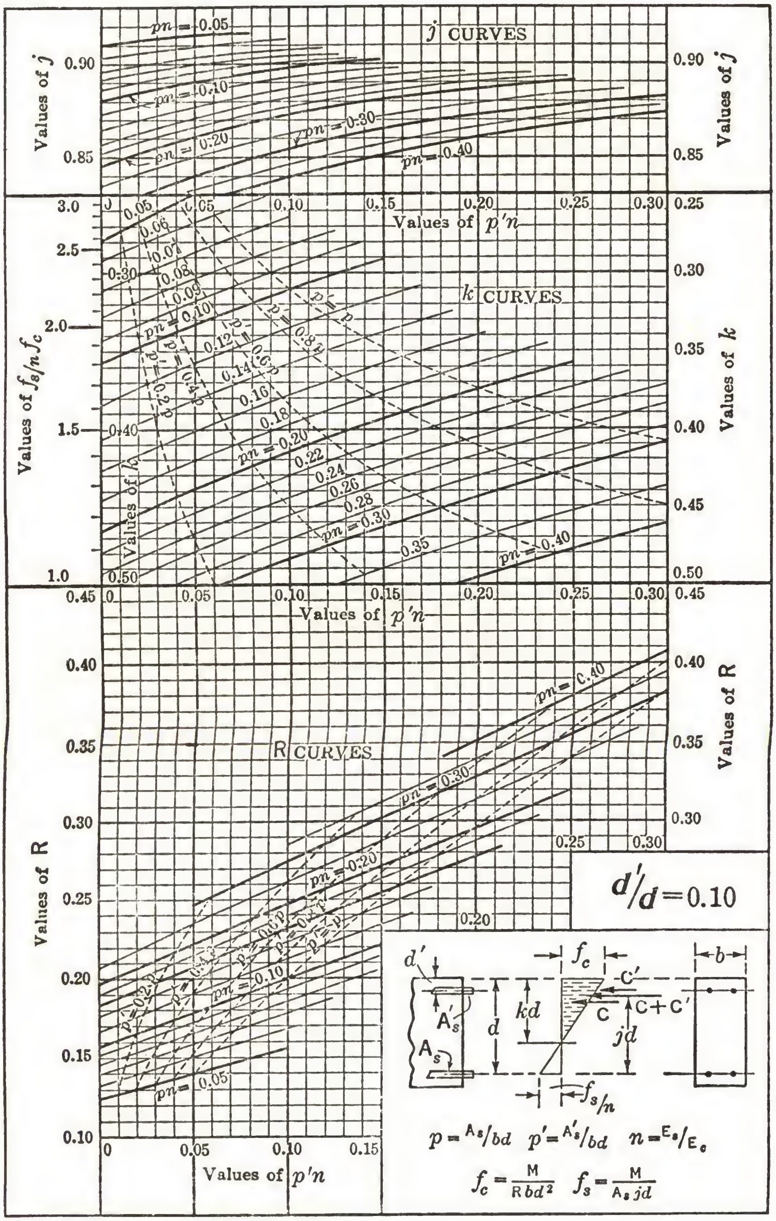
(Based upon the elastic theory with compressive reinforcement equal to (n + 1) times the displaced concrete.) See Note on page 45.



RECTANGULAR BEAMS REINFORCED FOR COMPRESSION. $M = f_c R b d^2$.

(WORKING STRESS METHOD)

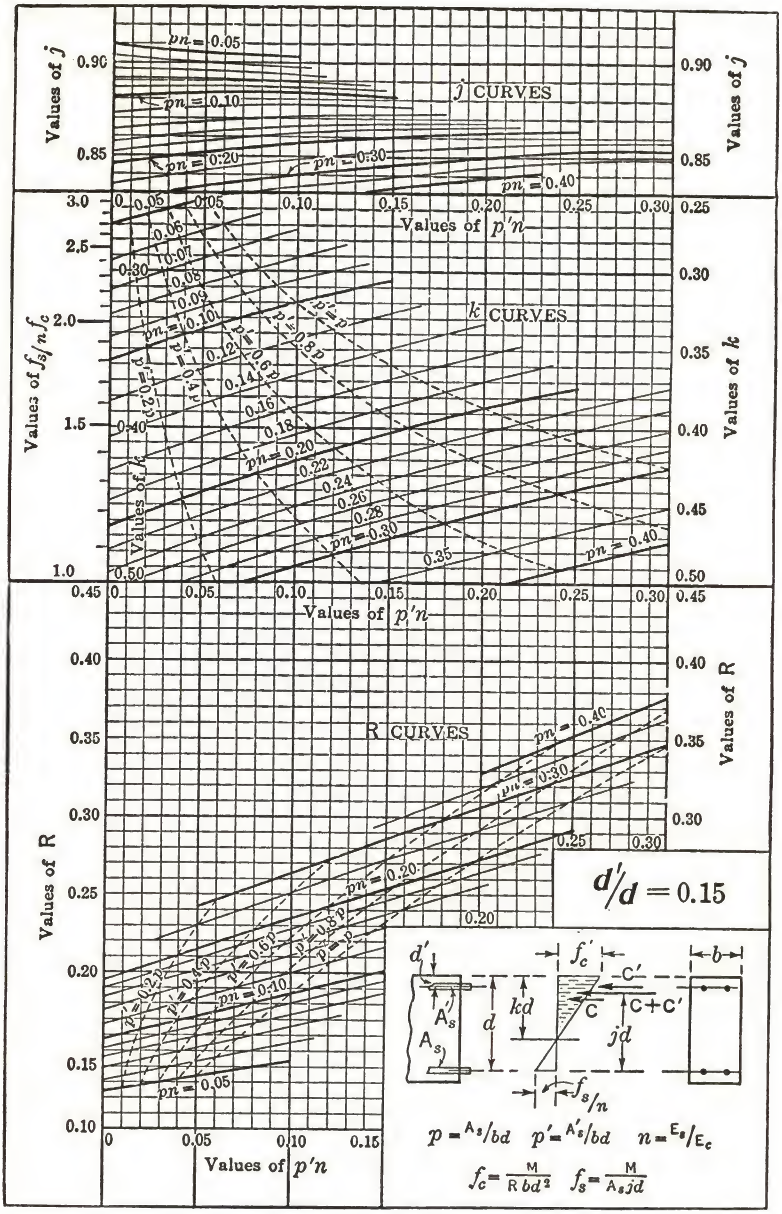
(Based upon the elastic theory with compressive reinforcement equal to (n + 1) times the displaced concrete.) **See Note on page 45.**



RECTANGULAR BEAMS REINFORCED FOR COMPRESSION. $M = f_cRbd^2$.

(WORKING STRESS METHOD)

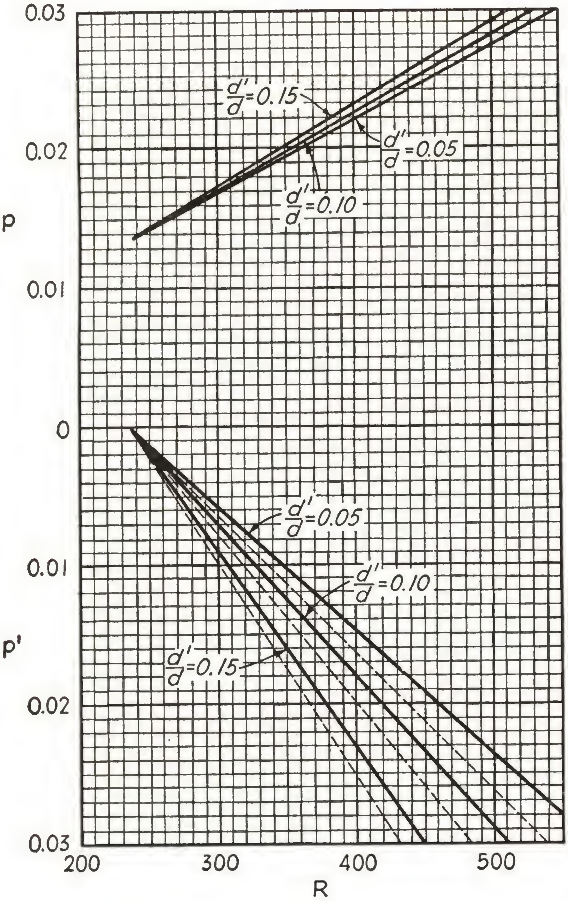
(Based upon the elastic theory with compressive reinforcement equal to (n + 1) times the displaced concrete.) See Note on page 45.



RECTANGULAR BEAMS REINFORCED FOR COMPRESSION. $M = Rbd^2$.

(WORKING STRESS METHOD)

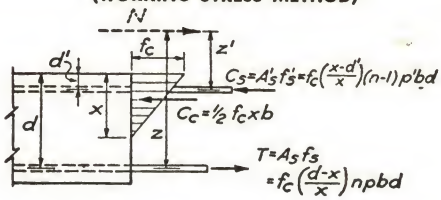
$f_c' = 3000$ psi
$f_c = 1350$ psi
$f_s = 20{,}000$ psi
$n = 10$



(Based upon the elastic theory.) The lower dotted lines employ n times the displaced concrete instead of $(n + 1)$. Theoretically, the steel must be replaced by n times as much concrete, part of which goes into the space occupied by the steel, leaving $(n - 1)A_s'$ in wings outside the beam section. If these wings are given an area nA_s' (as on pages 42-44), the steel is actually replaced by $(n + 1)$ times its area, a relatively minor refinement since the value of $n = \frac{E_s}{E_c}$ is rarely known to such a high degree of precision.

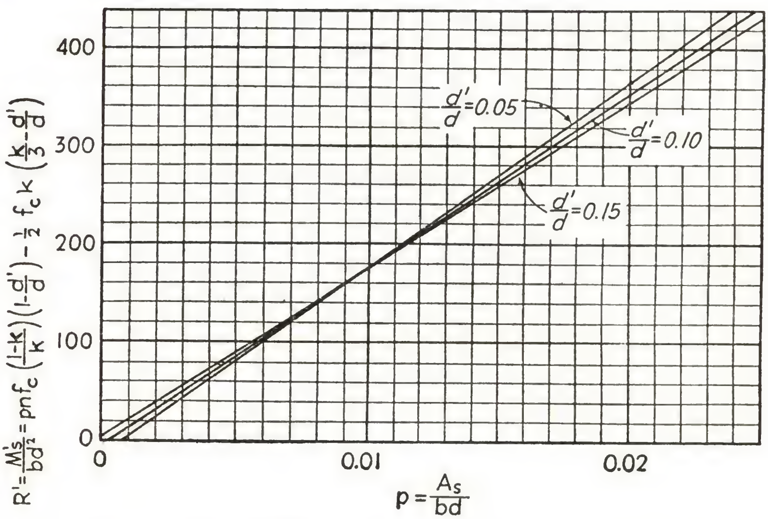
BENDING AND DIRECT COMPRESSION

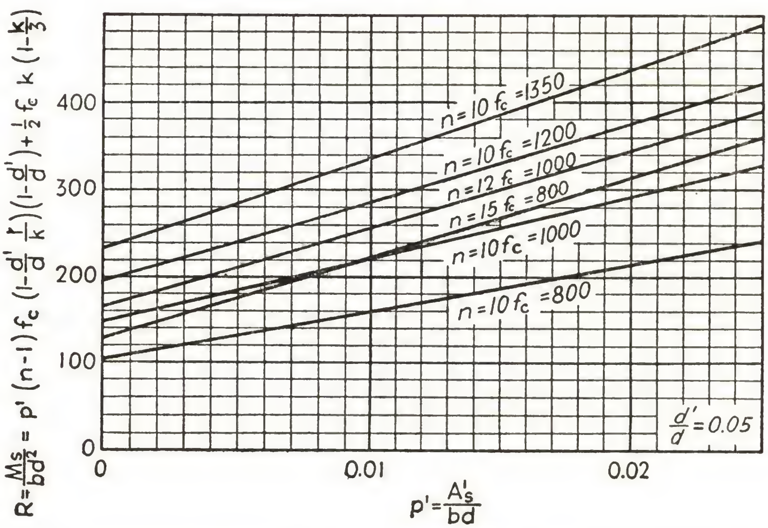
(WORKING STRESS METHOD)



Moment about Tension Steel: $Nz = M_s$; then $R = \frac{M_s}{bd^2}$, M_s being the moment about the tension steel and R being taken from the lower chart on this page or from page 47.

Moment about Compression Steel: $Nz' = M'_s$; then $R' = \frac{M'_s}{bd^2}$, M'_s being the moment about the compression steel and R being taken from the upper chart on this page.

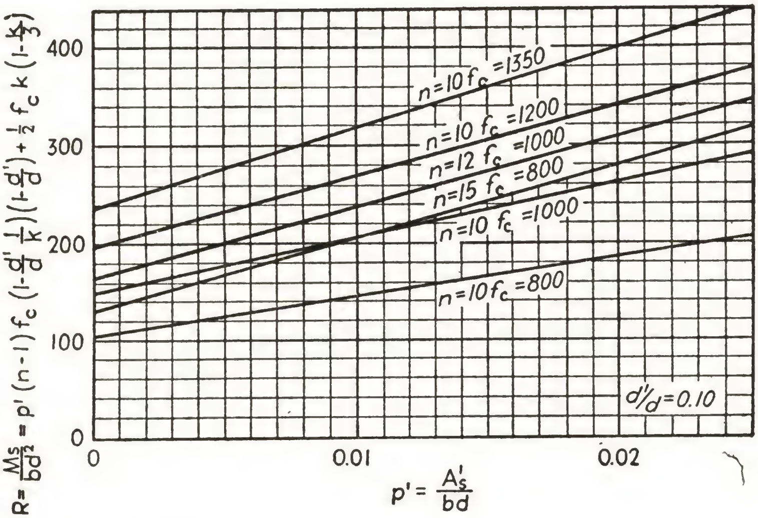




BENDING AND DIRECT COMPRESSION

(WORKING STRESS METHOD)

(See explanation on page 46.)



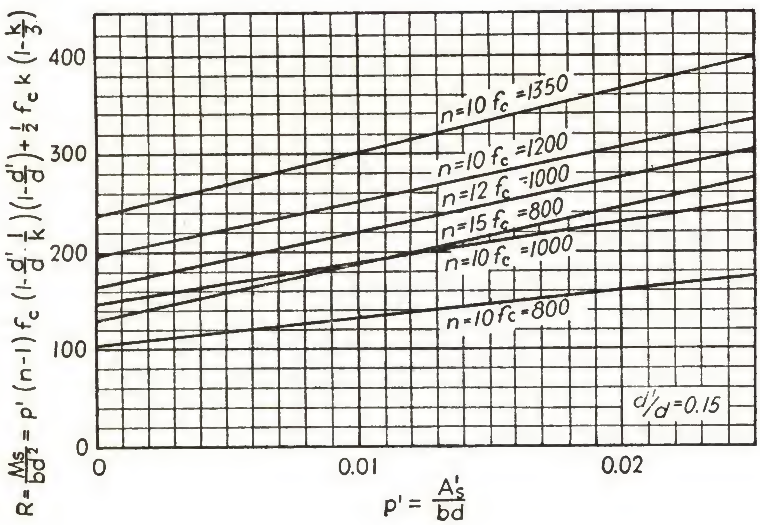
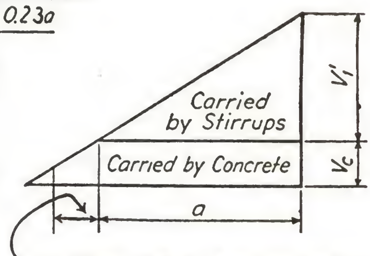


TABLE OF STIRRUP SPACINGS WITH TRIANGULAR SHEAR VARIATION.

Number of Stirrups at One End	Distance First Stirrup to Face of Support	Spacing, Center to Center, of Stirrups, in Terms of a									
		1st Group		2nd Group		3rd Group		4th Group		5th Group	
		No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing	No.	Spacing
20	0.013 a	8	0.03 a	7	0.04 a	2	0.06 a	1	0.08 a	1	0.11 a
19	0.013 a	7	0.03 a	6	0.04 a	3	0.06 a	1	0.08 a	1	0.12 a
18	0.014 a	6	0.03 a	5	0.04 a	4	0.06 a	1	0.08 a	1	0.12 a
17	0.015 a	5	0.03 a	5	0.04 a	4	0.06 a	1	0.09 a	1	0.13 a
16	0.016 a	3	0.03 a	5	0.04 a	5	0.06 a	1	0.09 a	1	0.13 a
15	0.017 a	2	0.03 a	5	0.04 a	4	0.06 a	2	0.08 a	1	0.14 a
14	0.018 a	5	0.04 a	4	0.05 a	2	0.08 a	1	0.09 a	1	0.14 a
13	0.019 a	4	0.04 a	3	0.05 a	3	0.08 a	1	0.09 a	1	0.14 a
12	0.021 a	6	0.05 a	3	0.07 a	1	0.12 a	1	0.15 a		
11	0.023 a	5	0.05 a	3	0.08 a	1	0.12 a	1	0.15 a		
10	0.025 a	3	0.05 a	4	0.08 a	1	0.12 a	1	0.16 a		
9	0.028 a	3	0.06 a	3	0.09 a	1	0.12 a	1	0.17 a		
8	0.032 a	2	0.07 a	3	0.09 a	1	0.13 a	1	0.18 a		
7	0.036 a	3	0.08 a	2	0.13 a	1	0.20 a				
6	0.04 a	3	0.10 a	1	0.15 a	1	0.22 a				
5	0.05 a	2	0.12 a	1	0.16 a	1	0.23 a				
4	0.07 a	2	0.16 a	1	0.26 a						
3	0.09 a	1	0.21 a	1	0.30 a						
2	0.13 a	1	0.37 a								
1	0.29 a										

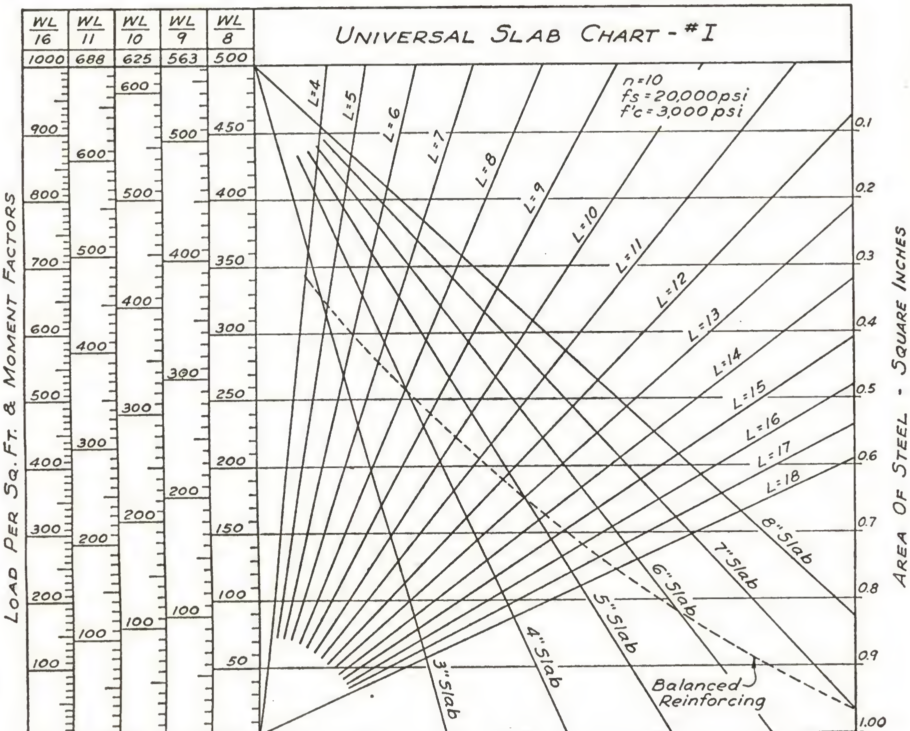


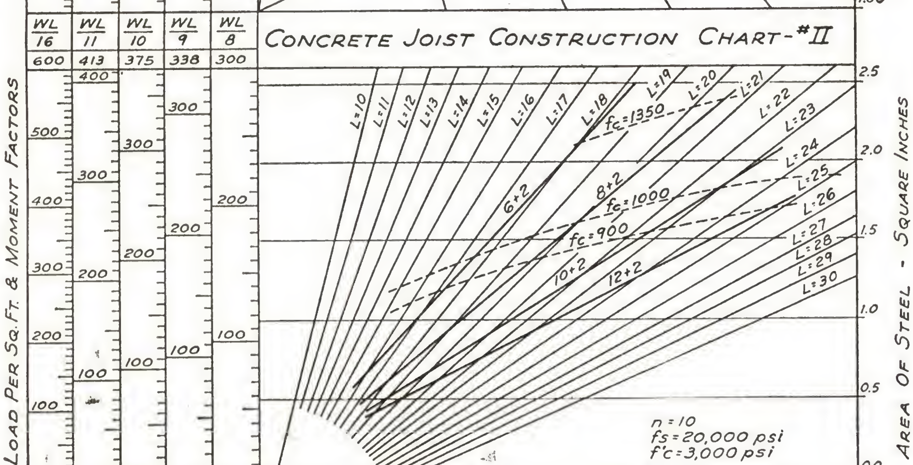
Stirrups are to be carried on for a distance "d" (one beam depth).

See explanation on pages 86 to 91.

SAFE LOAD CHARTS

The four charts on pages 49 and 50 are included not so much for design use as to show how a structural designer can prepare charts to solve almost any problem which involves the equating of groups of functions (such as bending moment to resisting moment or shear intensity to shear resistance). A series of lines radiating from one corner gives all values of one function (say, bending moment), while a series of lines radiating from an opposite corner gives all values of the other function (say, resisting moment). A line parallel to the proper coordinate axis from a point on one line across others equates the functions (bending moment to resisting moment), giving several choices of possible solutions.



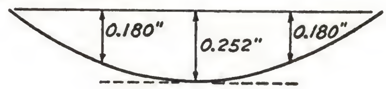


For Charts I, II and III 1. Enter on left with total load (psf) in column giving proper moment factor. 2. Horizontal to span in feet. 3. Vertical to desired depth. 4. Horizontal to steel area.

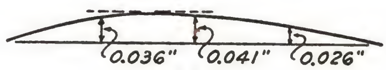
DEFLECTIONS OF REINFORCED CONCRETE BEAMS

Example III—What is the probable short-time deflection of the beam in Ex. I if a restraining moment of 200 ki is applied at the left end and 300 ki at the right?

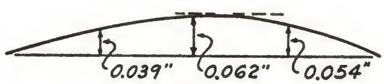
The uniform load produces downward deflection (*a* in fig.), maximum at mid-span. Left end moment produces upward deflection (*b*), maximum left of center; right end moment produces upward deflection (*c*), maximum right of center. A summation to determine absolute maximum might be complicated, but, in view of the many assumptions, it is sufficiently accurate to add deflections at center (and quarter-points) to obtain the curve (*d*) so:—



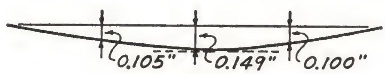
(a)



(b)



(c)



(d)

Δ for uniform load $= \dfrac{+WL^3}{76.8EI} = \dfrac{+1225 \times 20^4 \times 1728}{76.8 \times 3{,}000{,}000 \times 5832} = +0.252$ in.

Δ for 200^{ki} left end moment $= \dfrac{-3M_AL^{2*}}{48EI} = \dfrac{-3 \times 200{,}000 \times 20^2 \times 144}{48 \times 3{,}000{,}000 \times 5832} = -0.041$ in.

Δ for 300^{ki} right end moment $= \dfrac{-3M_BL^2}{48EI} = \dfrac{-3 \times 300{,}000 \times 20^2 \times 144}{48 \times 3{,}000{,}000 \times 5832} = \underline{-0.062 \text{ in.}}$

$+0.149$ in.

It is quite likely that a greater deflection would actually develop as discussed in Ex. IV.

Example IV—What is the probable maximum deflection of the beam of Ex. III at the end of a five year period if the live load and end moments are all fairly continuously in place?

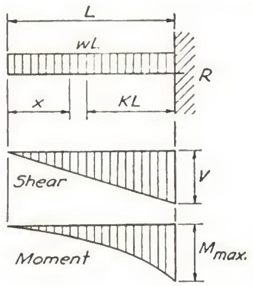
This example does not lend itself readily to simple mathematical analysis. The end slopes from moments computed by elastic frame methods, while reasonably accurate for short-time loading, might relieve themselves somewhat by creep in the concrete; certainly the deflection of the portion undergoing positive moment will be materially increased by such creep. Recent tests of continuous beams suggest that the long-time actual deflection can best be obtained by multiplying the short-time deflection by a factor, and 2½ at present seems a fair value for this multiplier. Hence, $2\frac{1}{2} \times 0.149 = 0.373$ in.

One of the most effective ways of reducing deflection caused by long-time loading is by the use of compressive reinforcement.

* This can be checked by the conjugate beam method, the M/EI diagram being a triangle from zero at the right end to M_A/EI at the left end. Right reaction is $\frac{1}{3} \cdot \frac{1}{2} \cdot \frac{M_AL}{EI}$, center ordinate is $\frac{M_A}{2EI}$ and moment of reaction less load is $\frac{1}{3} \cdot \frac{1}{2} \cdot \frac{M_AL}{EI} \cdot \frac{L}{2} - \frac{M_A}{2EI} \cdot \frac{L}{2} \cdot \frac{1}{2} \cdot \frac{1}{3} \cdot \frac{L}{2} = \frac{3M_AL^2}{48EI}$.

BEAM DIAGRAMS AND FORMULAS

CANTILEVERED BEAM—UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load.... $= 4wL$

$R = V$.................. $= wL$

V_x.................... $= wx$

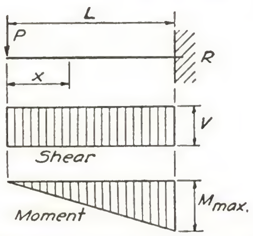
M_{max} (at fixed end)........ $= \frac{wL^2}{2}$

M_x.................... $= \frac{wx^2}{2}$

Δ_{max} (at free end)......... $= \frac{wL^4}{8EI}$

Δ_x.................... $= \frac{w}{24EI}(x^4 - 4L^3x + 3L^4)$

CANTILEVERED BEAM—CONCENTRATED LOAD AT FREE END



Equivalent Tabular Load..... $= 8P$

$R = V$.................. $= P$

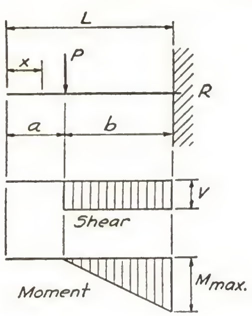
M_{max} (at fixed end)......... $= PL$

M_x.................... $= Px$

Δ_{max} (at free end).......... $= \frac{PL^3}{3EI}$

Δ_x.................... $= \frac{P}{6EI}(2L^3 - 3L^2x + x^3)$

CANTILEVERED BEAM—CONCENTRATED LOAD AT ANY POINT



Equivalent Tabular Load..... $= \frac{8Pb}{L}$

$R = V$ (when $x > a$)....... $= P$

M_{max} (at fixed end)....... $= Pb$

M_x (when $x > a$)........ $= P(x - a)$

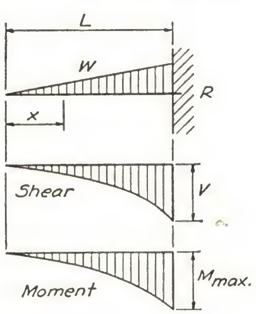
Δ_{max} (at free end)........ $= \frac{Pb^2}{6EI}(3L - b)$

Δa (at point of load)..... $= \frac{Pb^3}{3EI}$

Δ_x (when $x < a$)........ $= \frac{Pb^2}{6EI}(3L - 3x - b)$

Δ_x (when $x > a$)........ $= \frac{P(L - x)^2}{6EI}(3b - L + x)$

CANTILEVERED BEAM—LOAD INCREASING UNIFORMLY TO FIXED END



Equivalent Tabular Load........ $= \frac{8}{3}W$

$R = V$...................... $= W$

V_x........................ $= W\frac{x^2}{L^2}$

M_{max} (at fixed end)........... $= \frac{WL}{3}$

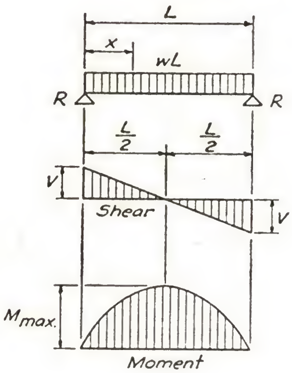
M_x........................ $= \frac{Wx^3}{3L^2}$

Δ_{max} (at free end)............ $= \frac{WL^3}{15EI}$

Δ_x........................ $= \frac{W}{60EIL^2}(x^5 - 5L^4x + 4L^5)$

BEAM DIAGRAMS AND FORMULAS

SIMPLE BEAM—UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load . . . . $= wL$

$R = V$ . . . . $= \dfrac{wL}{2}$

V_x . . . . $= w\left(\dfrac{L}{2} - x\right)$

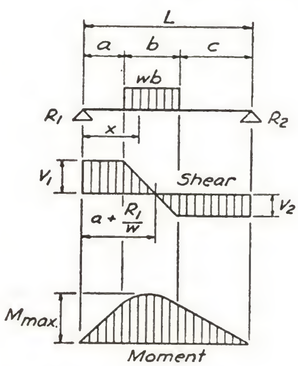
M_{max} (at center) . . . . $= \dfrac{wL^2}{8}$

M_x . . . . $= \dfrac{wx}{2}(L - x)$

Δ_{max} (at center) . . . . $= \dfrac{5wL^4}{384EI}$

Δ_x . . . . $= \dfrac{wx}{24EI}(L^3 - 2Lx^2 + x^3)$

SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED



$R_1 = V_1$ (max when $a < c$) . . . . . . $= \dfrac{wb}{2L}(2c + b)$

$R_2 = V_2$ (max when $a > c$) . . . . . . $= \dfrac{wb}{2L}(2a + b)$

V_x [when $x > a$ and $< (a + b)$] $= R_1 - w(x - a)$

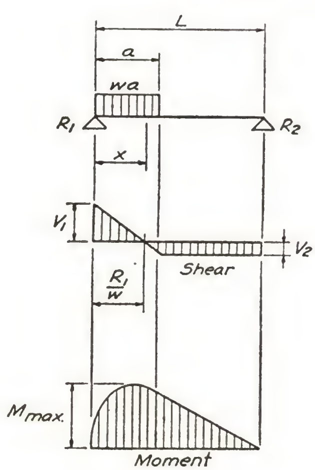
$M_{max}\left(\text{at } x = a + \dfrac{R_1}{w}\right)$ . . . . $= R_1\left(a + \dfrac{R_1}{2w}\right)$

M_x (when $x < a$) . . . . $= R_1x$

M_x [when $x > a$ and $< (a + b)$] $= R_1x - \dfrac{w}{2}(x - a)^2$

M_x [when $x > (a + b)$] . . . . $= R_2(L - x)$

SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



$R_1 = V_{1max}$ . . . . $= \dfrac{wa}{2L}(2L - a)$

$R_2 = V_2$ . . . . $= \dfrac{wa^2}{2L}$

V (when $x < a$) . . $= R_1 - wx$

$M_{max}\left(\text{at } x = \dfrac{R_1}{w}\right)$ . . $= \dfrac{R_1^2}{2w}$

M_x (when $x < a$) . . $= R_1x - \dfrac{wx^2}{2}$

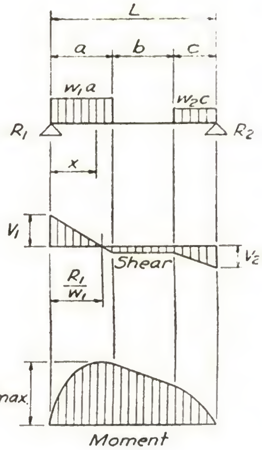
M_x (when $x > a$) . . $= R_2(L - x)$

Δ_x (when $x < a$) . . $= \dfrac{wx}{24EIL}[a^2(2L - a)^2 - 2ax^2(2L - a) + Lx^3]$

Δ_x (when $x > a$) . . $= \dfrac{wa^2(L - x)}{24EIL}(4xL - 2x^2 - a^2)$

BEAM DIAGRAMS AND FORMULAS

SIMPLE BEAM—UNIFORM LOAD PARTIALLY DISTRIBUTED AT EACH END



$R_1 = V_1$ .... $= \dfrac{w_1a(2L - a) + w_2c^2}{2L}$

$R_2 = V_2$ .... $= \dfrac{w_2c(2L - c) + w_1a^2}{2L}$

V_x (when $x < a$) .... $= R_1 - w_1x$

V_x [when $x > a$ and $<(a + b)$] ..... $= R_1 - w_1a$

V_x [when $x > (a + b)$] .... $= R_2 - w_2(L - x)$

M_{max} $\left(\text{at } x = \dfrac{R_1}{w_1} \text{ when } R_1 < w_1a\right)$ ..... $= \dfrac{R_1^2}{2w_1}$

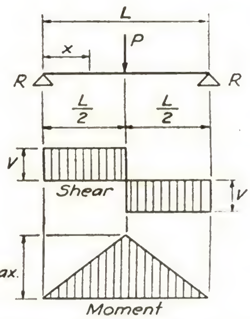
M_{max} $\left(\text{at } x = L - \dfrac{R_2}{w_2} \text{ when } R_2 < w_2c\right)$. $= \dfrac{R_2^2}{2w_2}$

M_x (when $x < a$) .... $= R_1x - \dfrac{w_1x^2}{2}$

M_x [when $x > a$ and $<(a + b)$] .... $= R_1x - \dfrac{w_1a}{2}(2x - a)$

M_x [when $x > (a + b)$] .... $= R_2(L - x) - \dfrac{w_2(L - x)^2}{2}$

SIMPLE BEAM—CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load .... $= 2P$

$R = V$ .... $= \dfrac{P}{2}$

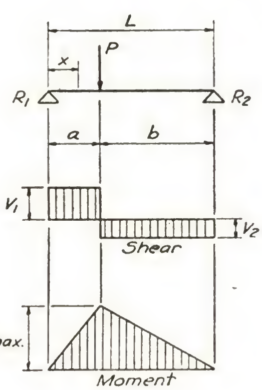
M_{max} (at point of load) .... $= \dfrac{PL}{4}$

M_x $\left(\text{when } x < \dfrac{L}{2}\right)$ .... $= \dfrac{Px}{2}$

Δ_{max} (at point of load) .... $= \dfrac{PL^3}{48EI}$

Δ_x $\left(\text{when } x < \dfrac{L}{2}\right)$ .... $= \dfrac{Px}{48EI}(3L^2 - 4x^2)$

SIMPLE BEAM—CONCENTRATED LOAD AT ANY POINT



Equivalent Tabular Load .... $= \dfrac{8Pab}{L^2}$

$R_1 = V_1$ (max when $a < b$) .... $= \dfrac{Pb}{L}$

$R_2 = V_2$ (max when $a > b$) .... $= \dfrac{Pa}{L}$

M_{max} (at point of load) .... $= \dfrac{Pab}{L}$

M_x (when $x < a$) .... $= \dfrac{Pbx}{L}$

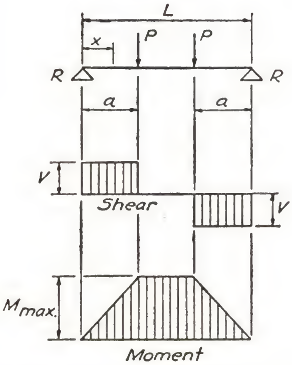
Δ_{max} $\left[\text{at } x = \sqrt{\dfrac{a(a + 2b)}{3}} \text{ when } a > b\right]$. $= \dfrac{Pab(a + 2b)\sqrt{3a(a + 2b)}}{27EIL}$

Δ_a (at point of load) .... $= \dfrac{Pa^2b^2}{3EIL}$

Δ_x (when $x < a$) .... $= \dfrac{Pbx}{6EIL}(L^2 - b^2 - x^2)$

BEAM DIAGRAMS AND FORMULAS

SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Equivalent Tabular Load. . . . . $= \dfrac{8Pa}{L}$

$R = V$ [when $x < a$, or $> (L - a)$]. . . . . . $= P$

M_{max} (between loads). . . . . $= Pa$

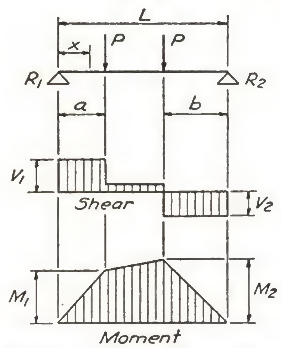
M_x (when $x < a$). . . . . $= Px$

Δ_{max} (at center). $= \dfrac{Pa}{24EI}(3L^2 - 4a^2)$

Δ_x (when $x < a$). . . . . $= \dfrac{Px}{6EI}(3La - 3a^2 - x^2)$

Δ_x [when $x > a$ and $< (L - a)$]. . . . . . . $= \dfrac{Pa}{6EI}(3Lx - 3x^2 - a^2)$

SIMPLE BEAM—TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ (max when $a < b$). . . . . $= \dfrac{P}{L}(L - a + b)$

$R_2 = V_2$ (max when $a > b$). . . . . $= \dfrac{P}{L}(L - b + a)$

V_x [when $x > a$ and $< (L - b)$]. . . . . . . $= \dfrac{P}{L}(b - a)$

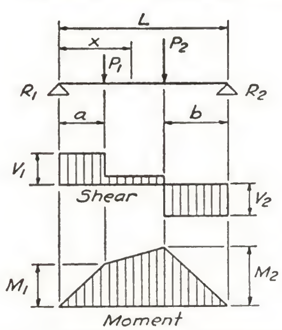
M_1 (max when $a > b$). . . . . $= R_1a$

M_2 (max when $a < b$). . . . . $= R_2b$

M_x (when $x < a$). . . . . $= R_1x$

M_x [when $x > a$ and $< (L - b)$]. . . . . . . $= R_1x - P(x - a)$

SIMPLE BEAM—TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$. . . . . $= \dfrac{P_1(L - a) + P_2b}{L}$

$R_2 = V_2$. . . . . $= \dfrac{P_1a + P_2(L - b)}{L}$

V_x [when $x > a$ and $< (L - b)$]. . . . . $= R_1 - P_1$

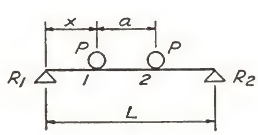
M_1 (max when $R_1 < P_1$). . . . . $= R_1a$

M_2 (max when $R_2 < P_2$). . . . . $= R_2b$

M_x (when $x < a$). . . . . $= R_1x$

M_x [when $x > a$ and $< (L - b)$]. . . . . $= R_1x - P_1(x - a)$

SIMPLE BEAM—TWO EQUAL CONCENTRATED MOVING LOADS

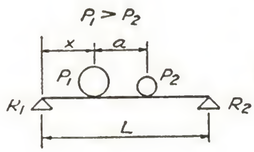


$R_{1max} = V_{1max}$ (at $x = 0$). . . . . $= P\left(2 - \dfrac{a}{L}\right)$

M_{max} $\begin{cases} \left[\begin{array}{l}\text{when } a < (2 - \sqrt{2})L = .586L \\ \text{under load 1 at } x = \dfrac{1}{2}\left(L - \dfrac{a}{2}\right)\end{array}\right] \ldots = \dfrac{P}{2L}\left(L - \dfrac{a}{2}\right)^2 \\ \left[\begin{array}{l}\text{when } a > (2 - \sqrt{2})L = .586L \\ \text{with one load at center of span}\end{array}\right] \ldots = \dfrac{PL}{4}\end{cases}$

BEAM DIAGRAMS AND FORMULAS

SIMPLE BEAM—TWO UNEQUAL CONCENTRATED MOVING LOAD

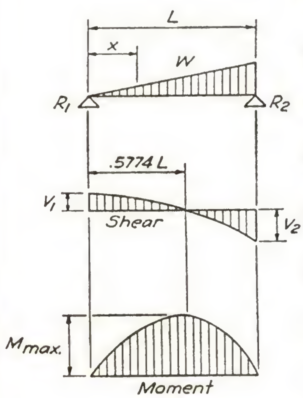


$R_{1max} = V_{1max}$ (at $x = 0$) .... $= P_1 + P_2\left(\frac{L-a}{L}\right)$

M_{max}:

- $\left[\text{under } P_1 \text{ at } x = \frac{1}{2}\left(L - \frac{P_2 a}{P_1 + P_2}\right)\right]$ .. $= (P_1 + P_2)\frac{x^2}{L}$
- $\left[\right.$ M_{max} may occur with larger load at center of span and other load off span $\left.\right]$ .. $= \frac{P_1 L}{4}$

SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO ONE END



Equivalent Tabular Load .... $= \frac{16W}{9\sqrt{3}} = 1.0264W$

$R_1 = V_1$ .... $= \frac{W}{3}$

$R_2 = V_2 max$ .... $= \frac{2W}{3}$

V_x .... $= \frac{W}{3} - \frac{Wx^2}{L^2}$

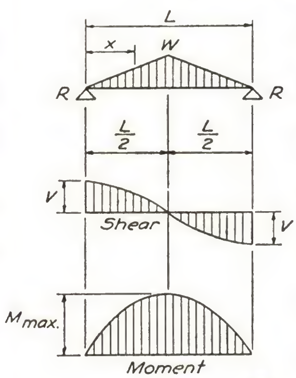
$M_{max}\left(\text{at } x = \frac{L}{\sqrt{3}} = .5774L\right)$ .... $= \frac{2WL}{9\sqrt{3}} = .1283WL$

M_x .... $= \frac{Wx}{3L^2}(L^2 - x^2)$

$\Delta max\left(\text{at } x = L\sqrt{1 - \sqrt{\frac{8}{15}}} = .5193L\right)$. $= .01304\frac{WL^3}{EI}$

Δx .... $= \frac{Wx}{180EIL^2}(3x^4 - 10L^2x^2 + 7L^4)$

SIMPLE BEAM—LOAD INCREASING UNIFORMLY TO CENTER



Equivalent Tabular Load .... $= \frac{4W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x $\left(\text{when } x < \frac{L}{2}\right)$ .... $= \frac{W}{2L^2}(L^2 - 4x^2)$

M_{max} (at center) .... $= \frac{WL}{6}$

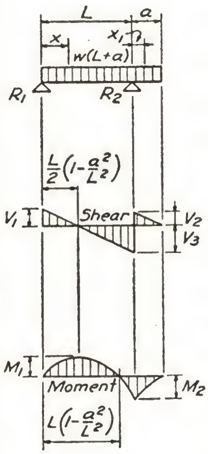
M_x $\left(\text{when } x < \frac{L}{2}\right)$ .... $= Wx\left(\frac{1}{2} - \frac{2x^2}{3L^2}\right)$

Δmax (at center) .... $= \frac{WL^3}{60EI}$

Δx .... $= \frac{Wx}{480EIL^2}(5L^2 - 4x^2)^2$

BEAM DIAGRAMS AND FORMULAS

BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1 \ldots\ldots = \frac{w}{2L}(L^2 - a^2)$

$R_2 = V_2 + V_3 \ldots\ldots = \frac{w}{2L}(L + a)^2$

$V_2 \ldots\ldots = wa$

$V_3 \ldots\ldots = \frac{w}{2L}(L^2 + a^2)$

V_x (between supports) $\ldots\ldots = R_1 - wx$

V_{x_1} (for overhang) $\ldots\ldots = w(a - x_1)$

$M_1 \left(\text{at } x = \frac{L}{2}\left[1 - \frac{a^2}{L^2}\right]\right) \ldots\ldots = \frac{w}{8L^2}(L + a)^2(L - a)^2$

M_2 (at R_2) $\ldots\ldots = \frac{wa^2}{2}$

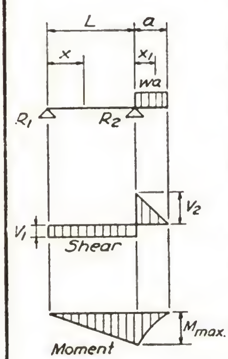
M_x (between supports) $\ldots\ldots = \frac{wx}{2L}(L^2 - a^2 - xL)$

M_{x_1} (for overhang) $\ldots\ldots = \frac{w}{2}(a - x_1)^2$

Δ_x (between supports) $\ldots\ldots = \frac{wx}{24EIL}(L^4 - 2L^2x^2 + Lx^3 - 2a^2L^2 + 2a^2x^2)$

Δ_{x_1} (for overhang) $\ldots\ldots = \frac{wx_1}{24EI}(4a^2L - L^3 + 6a^2x_1 - 4ax_1^2 + x_1^3)$

BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD ON OVERHANG



$R_1 = V_1 \ldots\ldots = \frac{wa^2}{2L}$

$R_2 = V_1 + V_2 \ldots\ldots = \frac{wa}{2L}(2L + a)$

$V_2 \ldots\ldots = wa$

V_{x_1} (for overhang) $\ldots\ldots = w(a - x_1)$

M_{max} (at R_2) $\ldots\ldots = \frac{wa^2}{2}$

M_x (between supports) $\ldots\ldots = \frac{wa^2x}{2L}$

M_{x_1} (for overhang) $\ldots\ldots = \frac{w}{2}(a - x_1)^2$

$\Delta_{max} \left(\text{between supports at } x = \frac{L}{\sqrt{3}}\right) \ldots = \frac{wa^2L^2}{18\sqrt{3}EI} = .03208\frac{wa^2L^2}{EI}$

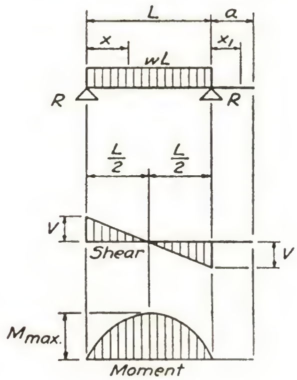
Δ_{max} (for overhang at $x_1 = a$) $\ldots\ldots = \frac{wa^3}{24EI}(4L + 3a)$

Δx (between supports) $\ldots\ldots = \frac{wa^2x}{12EIL}(L^2 - x^2)$

Δx_1 (for overhang) $\ldots\ldots = \frac{wx_1}{24EI}(4a^2L + 6a^2x_1 - 4ax_1^2 + x_1^3)$

BEAM DIAGRAMS AND FORMULAS

BEAM OVERHANGING ONE SUPPORT—UNIFORMLY DISTRIBUTED LOAD BETWEEN SUPPORTS



Equivalent Tabular Load .... $= wL$

$R = V$ .... $= \frac{wL}{2}$

V_x .... $= w\left(\frac{L}{2} - x\right)$

M_{max} (at center) .... $= \frac{wL^2}{8}$

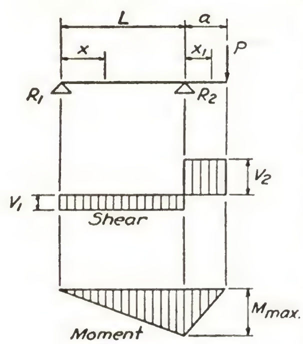
M_x .... $= \frac{wx}{2}(L - x)$

Δ_{max} (at center) .... $= \frac{5wL^4}{384EI}$

Δx .... $= \frac{wx}{24EI}(L^3 - 2Lx^2 + x^3)$

Δx_1 .... $= \frac{wL^3x_1}{24EI}$

BEAM OVERHANGING ONE SUPPORT—CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1$ .... $= \frac{Pa}{L}$

$R_2 = V_1 + V_2$ .... $= \frac{P}{L}(L + a)$

V_2 .... $= P$

M_{max} (at R_2) .... $= Pa$

M_x (between supports) .... $= \frac{Pax}{L}$

M_{x_1} (for overhang) .... $= P(a - x_1)$

Δ_{max} $\left(\text{between supports at } x = \frac{L}{\sqrt{3}}\right)$ ... $= \frac{PaL^2}{9\sqrt{3}EI} = .06415\frac{PaL^2}{EI}$

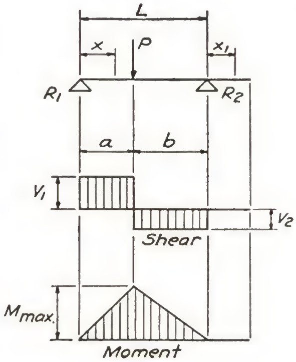
Δ_{max} (for overhang at $x_1 = a$) .... $= \frac{Pa^2}{3EI}(L + a)$

Δx (between supports) .... $= \frac{Pax}{6EIL}(L^2 - x^2)$

Δx_1 (for overhang) .... $= \frac{Px_1}{6EI}(2aL + 3ax_1 - x_1^2)$

BEAM DIAGRAMS AND FORMULAS

BEAM OVERHANGING ONE SUPPORT—CONCENTRATED LOAD AT ANY POINT BETWEEN SUPPORTS



Equivalent Tabular Load . . . . $= \dfrac{8Pab}{L^2}$

$R_1 = V_1$ (max when $a < b$) . . . . $= \dfrac{Pb}{L}$

$R_2 = V_2$ (max when $a > b$) . . . . $= \dfrac{Pa}{L}$

M_{max} (at point of load) . . . . $= \dfrac{Pab}{L}$

M_x (when $x < a$) . . . . $= \dfrac{Pbx}{L}$

Δ_{max} $\left[\text{at } x = \sqrt{\dfrac{a(a+2b)}{3}} \text{ when } a > b\right]$. $= \dfrac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIL}$

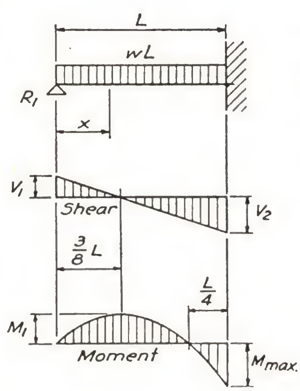
Δa (at point of load) . . . . $= \dfrac{Pa^2b^2}{3EIL}$

Δx (when $x < a$) . . . . $= \dfrac{Pbx}{6EIL}(L^2 - b^2 - x^2)$

Δx (when $x > a$) . . . . $= \dfrac{Pa(L-x)}{6EIL}(2Lx - x^2 - a^2)$

Δx_1 . . . . $= \dfrac{Pabx_1}{6EIL}(L+a)$

BEAM FIXED AT ONE END, SUPPORTED AT OTHER—UNIFORMLY DISTRIBUTED LOAD



Equivalent Tabular Load . . . . $= wL$

$R_1 = V_1$ . . . . $= \dfrac{3wL}{8}$

$R_2 = V_2max$ . . . . $= \dfrac{5wL}{8}$

V_x . . . . $= R_1 - wx$

M_{max} . . . . $= \dfrac{wL^2}{8}$

M_1 $\left(\text{at } x = \dfrac{3}{8}L\right)$ . . . . $= \dfrac{9}{128}wL^2$

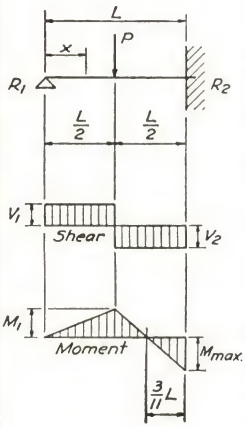
M_x . . . . $= R_1x - \dfrac{wx^2}{2}$

$\Delta max\left[\text{at } x = \dfrac{L}{16}(1+\sqrt{33}) = .4215L\right]$. . $= \dfrac{wL^4}{184.63EI}$

Δx . . . . $= \dfrac{wx}{48EI}(L^3 - 3Lx^2 + 2x^3)$

BEAM DIAGRAMS AND FORMULAS

BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load . . . . $= \frac{3P}{2}$

$R_1 = V_1$ . . . . $= \frac{5P}{16}$

$R_2 = V_2 max$ . . . . $= \frac{11P}{16}$

M_{max} (at fixed end) . . . . $= \frac{3PL}{16}$

M_1 (at point of load) . . . . $= \frac{5PL}{32}$

M_x $\left(\text{when } x < \frac{L}{2}\right)$ . . . . $= \frac{5Px}{16}$

M_x $\left(\text{when } x > \frac{L}{2}\right)$ . . . . $= P\left(\frac{L}{2} - \frac{11x}{16}\right)$

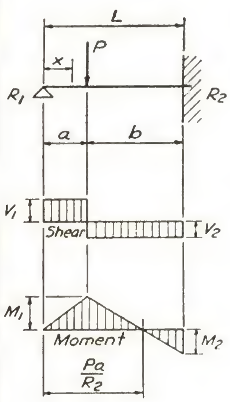
Δ_{max} $\left(\text{at } x = L\sqrt{\frac{1}{5}} = .4472L\right)$ . . . . $= \frac{PL^3}{48EI\sqrt{5}} = .009317\frac{PL^3}{EI}$

Δx (at point of load) . . . . $= \frac{7PL^3}{768EI}$

Δx $\left(\text{when } x < \frac{L}{2}\right)$ . . . . $= \frac{Px}{96EI}(3L^2 - 5x^2)$

Δx $\left(\text{when } x > \frac{L}{2}\right)$ . . . . $= \frac{P}{96EI}(x - L)^2(11x - 2L)$

BEAM FIXED AT ONE END, SUPPORTED AT OTHER—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ . . . . $= \frac{Pb^2}{2L^3}(a + 2L)$

$R_2 = V_2$ . . . . $= \frac{Pa}{2L^3}(3L^2 - a^2)$

M_1 (at point of load) . . . . $= R_1 a$

M_2 (at fixed end) . . . . $= \frac{Pab}{2L^2}(a + L)$

M_x (when $x < a$) . . . . $= R_1 x$

M_x (when $x > a$) . . . . $= R_1 x - P(x - a)$

Δ_{max} $\left(\text{when } a < .414L \text{ at } x = L\frac{L^2 + a^2}{3L^2 - a^2}\right)$ . . $= \frac{Pa(L^2 - a^2)^3}{3EI(3L^2 - a^2)^2}$

Δ_{max} $\left(\text{when } a > .414L \text{ at } x = L\sqrt{\frac{a}{2L + a}}\right)$. $= \frac{Pab^2}{6EI}\sqrt{\frac{a}{2L + a}}$

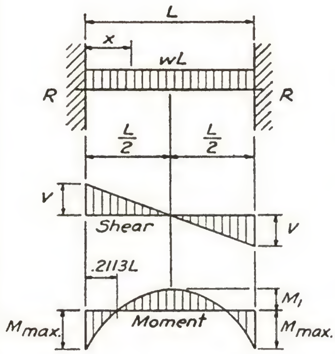
Δa (at point of load) . . . . $= \frac{Pa^2b^3}{12EIL^3}(3L + a)$

Δx (when $x < a$) . . . . $= \frac{Pb^2x}{12EIL^3}(3aL^2 - 2Lx^2 - ax^2)$

Δx (when $x > a$) . . . . $= \frac{Pa}{12EIL^3}(L - x)^2(3L^2x - a^2x - 2a^2L)$

BEAM DIAGRAMS AND FORMULAS

BEAM FIXED AT BOTH ENDS—UNIFORMLY DISTRIBUTED LOADS



Equivalent Tabular Load .... $= \frac{2wL}{3}$

$R = V$ .... $= \frac{wL}{2}$

V_x .... $= w\left(\frac{L}{2} - x\right)$

M_{max} (at ends) .... $= \frac{wL^2}{12}$

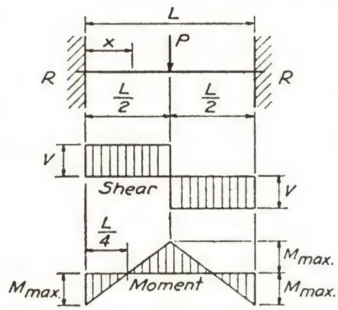
M_1 (at center) .... $= \frac{wL^2}{24}$

M_x .... $= \frac{w}{12}(6Lx - L^2 - 6x^2)$

Δ_{max} (at center) .... $= \frac{wL^4}{384EI}$

Δx .... $= \frac{wx^2}{24EI}(L - x)^2$

BEAM FIXED AT BOTH ENDS—CONCENTRATED LOAD AT CENTER



Equivalent Tabular Load .... $= P$

$R = V$ .... $= \frac{P}{2}$

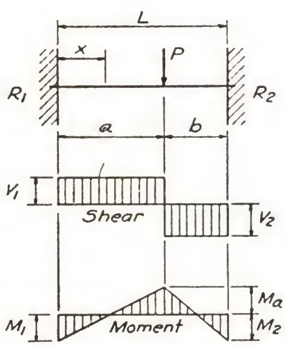
M_{max} (at center and ends) .... $= \frac{PL}{8}$

M_x $\left(\text{when } x < \frac{L}{2}\right)$ .... $= \frac{P}{8}(4x - L)$

Δ_{max} (at center) .... $= \frac{PL^3}{192EI}$

Δx $\left(x < \frac{L}{2}\right)$ .... $= \frac{Px^2}{48EI}(3L - 4x)$

BEAM FIXED AT BOTH ENDS—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ (max when $a < b$) .... $= \frac{Pb^2}{L^3}(3a + b)$

$R_2 = V_2$ (max when $a > b$) .... $= \frac{Pa^2}{L^3}(a + 3b)$

M_1 (max when $a < b$) .... $= \frac{Pab^2}{L^2}$

M_2 (max when $a > b$) .... $= \frac{Pa^2b}{L^2}$

M_a (at point of load) .... $= \frac{2Pa^2b^2}{L^3}$

M_x (when $x < a$) .... $= R_1x - \frac{Pab^2}{L^2}$

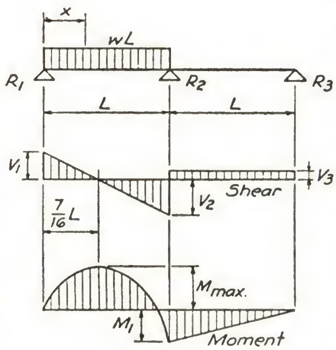
Δ_{max} $\left(\text{when } a > b \text{ at } x = \frac{2aL}{3a + b}\right)$ ... $= \frac{2Pa^3b^2}{3EI(3a + b)^2}$

Δa (at point of load) .... $= \frac{Pa^3b^3}{3EIL^3}$

Δx (when $x < a$) .... $= \frac{Pb^2x^2}{6EIL^3}(3aL - 3ax - bx)$

BEAM DIAGRAMS AND FORMULAS

CONTINUOUS BEAM—TWO EQUAL SPANS—UNIFORM LOAD ON ONE SPAN



Equivalent Tabular Load.............. $= \frac{49}{64}wL$

$R_1 = V_1$.......................... $= \frac{7}{16}wL$

$R_2 = V_2 + V_3$..................... $= \frac{5}{8}wL$

$R_3 = V_3$.......................... $= -\frac{1}{16}wL$

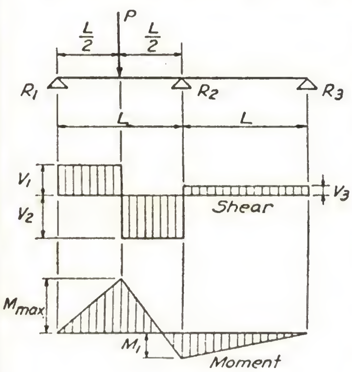
V_2.............................. $= \frac{9}{16}wL$

$M_{max}\left(\text{at } x = \frac{7}{16}L\right)$.............. $= \frac{49}{512}wL^2$

M_1 (at support R_2)................ $= \frac{1}{16}wL^2$

M_x (when $x < L$)................. $= \frac{wx}{16}(7L - 8x)$

CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT CENTER OF ONE SPAN



Equivalent Tabular Load.............. $= \frac{13}{8}P$

$R_1 = V_1$.......................... $= \frac{13}{32}P$

$R_2 = V_2 + V_3$..................... $= \frac{11}{16}P$

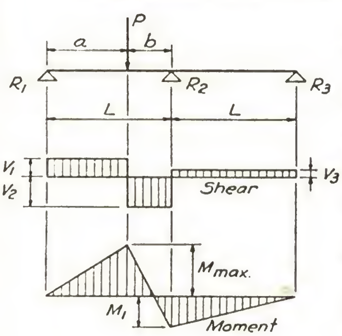
$R_3 = V_3$.......................... $= -\frac{3}{32}P$

V_2.............................. $= \frac{19}{32}P$

M_{max} (at point of load)............... $= \frac{13}{64}PL$

M_1 (at support R_2)................ $= \frac{3}{32}PL$

CONTINUOUS BEAM—TWO EQUAL SPANS—CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$.......................... $= \frac{Pb}{4L^3}[4L^2 - a(L + a)]$

$R_2 = V_2 + V_3$..................... $= \frac{Pa}{2L^3}[2L^2 + b(L + a)]$

$R_3 = V_3$.......................... $= -\frac{Pab}{4L^3}(L + a)$

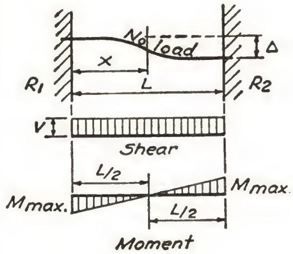
V_2.............................. $= \frac{Pa}{4L^3}[4L^2 + b(L + a)]$

M_{max} (at point of load)............... $= \frac{Pab}{4L^3}[4L^2 - a(L + a)]$

M_1 (at support R_2)................ $= \frac{Pab}{4L^2}(L + a)$

BEAM DIAGRAMS AND FORMULAS

BEAM FIXED BOTH ENDS—NO LOAD—ONE END OFFSET



$R_1 = V \ldots\ldots = \frac{12EI\Delta}{L^3}$

$R_2 \ldots\ldots = -\frac{12EI\Delta}{L^3}$

$M_{max} \ldots\ldots = \frac{6EI\Delta}{L^2}$

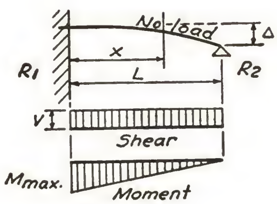
$M_x \ldots\ldots = \frac{6EI\Delta}{L^3}(2x - L)$

$\Delta_{max} \ldots\ldots = \Delta$

$\Delta_x \ldots\ldots = \frac{\Delta x^2}{L^3}(3L - 2x)$

(For offset in opposite direction reverse signs)

BEAM FIXED ONE END—NO LOAD—ONE END OFFSET



$R_1 = V \ldots\ldots = \frac{3EI\Delta}{L^3}$

$R_2 \ldots\ldots = -\frac{3EI\Delta}{L^3}$

$M_{max} \ldots\ldots = -\frac{3EI\Delta}{L^2}$

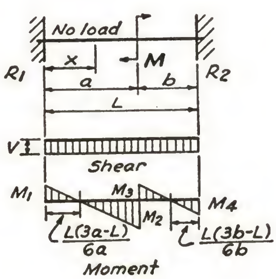
$M_x \ldots\ldots = -\frac{3EI\Delta}{L^3}(L - x)$

$\Delta_{max} \ldots\ldots = \Delta$

$\Delta_x \ldots\ldots = \frac{\Delta x^2}{2L^3}(3L - x)$

(For offset in opposite direction reverse signs)

BEAM FIXED BOTH ENDS—NO LOAD—APPLIED MOMENT



$R_1 = V = -\frac{6Mab}{L^3}$ $\qquad R_2 = \frac{6Mab}{L^3}$

$M_1 \ldots\ldots = -\frac{Mb}{L}\left(1 - 3\frac{a}{L}\right)$ $\qquad M_2 = -\frac{Mb}{L}\left(1 - 3\frac{a}{L} + 6\frac{a^2}{L^2}\right)$

$M_3 \ldots\ldots = \frac{Ma}{L}\left(1 - 3\frac{b}{L} + 6\frac{b^2}{L^2}\right)$ $\qquad M_4 = \frac{Ma}{L}\left(1 - 3\frac{b}{L}\right)$

M_x (when $x < a$)...... $= -\frac{Mb}{L^3}(L^2 - 3aL + 6ax)$

M_x (when $x > a$)...... $= \frac{Ma}{L^3}(L^2 + 3bL - 6bx)$

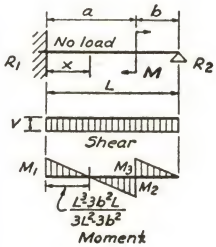
Δ_x (when $x < a$)...... $= \frac{Mb}{EIL^3}\left(\frac{L^2x^2}{2} - \frac{3aLx^2}{2} + ax^3\right)$

Δ_x (when $x > a$)...... $= -\frac{Ma}{EIL^3}\left(\frac{L^2x^2}{2} + \frac{3bLx^2}{2} - bx^3 - L^3x + \frac{aL^3}{2}\right)$

$\Delta_a \ldots\ldots = \frac{Ma^2b}{EIL^3}\left(\frac{L^2}{2} - \frac{3aL}{2} + a^2\right) = \frac{Ma^2b^2}{2EIL^3}(L - 2a)$

BEAM DIAGRAMS AND FORMULAS

BEAM FIXED ONE END—NO LOAD—APPLIED MOMENT



$R_1 = V \ldots\ldots = -\dfrac{3M}{2L}\left(1 - \dfrac{b^2}{L^2}\right)$ $\qquad R_2 \ldots\ldots = \dfrac{3M}{2L}\left(1 - \dfrac{b^2}{L^2}\right)$

$M_1 \ldots\ldots\ldots = \dfrac{M}{2}\left(1 - 3\dfrac{b^2}{L^2}\right)$ $\qquad M_2 \ldots\ldots = \dfrac{3Mb}{2L}\left(1 - \dfrac{b^2}{L^2}\right) - M$

$M_3 \ldots\ldots\ldots = \dfrac{3Mb}{2L}\left(1 - \dfrac{b^2}{L^2}\right)$

$M_x \text{ (when } x < a) \ldots = \dfrac{3M}{2L}\left(1 - \dfrac{b^2}{L^2}\right)(L - x) - M$

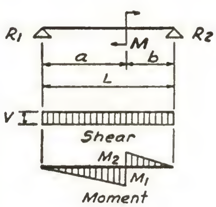
$M_x \text{ (when } x > a) \ldots = \dfrac{3M}{2L}\left(1 - \dfrac{b^2}{L^2}\right)(L - x)$

$\Delta_x \text{ (when } x < a) \ldots = \dfrac{Mx^2}{2EI}\left[\dfrac{3}{L}\left(1 - \dfrac{b^2}{L^2}\right)\left(\dfrac{x}{6} - \dfrac{L}{2}\right) + 1\right]$

$\Delta_x \text{ (when } x > a) \ldots = \dfrac{3M}{2EIL}\left(1 - \dfrac{b^2}{L^2}\right)\left(\dfrac{L^3}{3} - \dfrac{Lx^2}{2} + \dfrac{x^3}{6}\right) - \dfrac{Ma}{EI}(L - x)$

$\Delta_a \ldots\ldots\ldots\ldots\ldots = \dfrac{Ma^2}{2EI}\left[\dfrac{a(2L - a)(a - 3L)}{2L^3} + 1\right]$

SIMPLE BEAM—NO LOAD—APPLIED MOMENT



$R_1 = V \ldots\ldots\ldots\ldots = -\dfrac{M}{L}$ $\qquad R_2 \ldots\ldots\ldots\ldots\ldots\ldots = \dfrac{M}{L}$

$M_1 \ldots\ldots\ldots\ldots\ldots = -\dfrac{Ma}{L}$ $\qquad M_2 \ldots\ldots\ldots\ldots\ldots\ldots = \dfrac{Mb}{L}$

$M_{max} \ldots\ldots\ldots\ldots\ldots = M_1 \text{ if } a > b$

$\qquad = M_2 \text{ if } a < b$

$M_x \text{ (when } x < a) \ldots\ldots = -\dfrac{Mx}{L}$

$M_x \text{ (when } x > a) \ldots\ldots = \dfrac{M}{L}(L - x)$

$\Delta_x \text{ (when } x < a) \ldots\ldots = \dfrac{Mx}{6EIL^2}(-a^3 - 3a^2b + 2b^3 + Lx^2)$

$\Delta_x \text{ (when } x > a) \ldots\ldots = \dfrac{M(L - x)}{6EIL^2}[-2a^3 + 3ab^2 + b^3 - L(L - x)^2]$

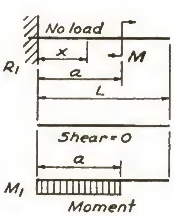
$\Delta_{max}\left(\text{if } a > b, \text{ at } x = \sqrt{-\dfrac{2}{3}L^2 + 2aL - a^2}\right) =$

$\dfrac{M\sqrt{-\dfrac{2}{3}L^2 + 2aL - a^2}}{6EIL^2}\left(-a^3 - 3a^2b + 2b^3 - \dfrac{2}{3}L^3 + 2aL^2 - a^2L\right)$

$\Delta_{max}\left(\text{if } a < b, \text{ at } (L - x) = \sqrt{-\dfrac{2}{3}L^2 + 2bL - b^2}\right) =$

$\dfrac{M\sqrt{-\dfrac{2L^2}{3} + 2bL - b^2}}{6EIL^2}\left(-2a^3 + 3ab^2 + b^3 + \dfrac{2}{3}L^3 - 2bL^2 + b^2L\right)$

CANTILEVERED BEAM—NO LOAD—APPLIED MOMENT



$R = V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = 0$

$M_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots = -M$

$\Delta_x \text{ (when } x \lesseqgtr a) \ldots\ldots\ldots\ldots\ldots\ldots\ldots = \dfrac{Mx^2}{2EI}$

$\Delta_x \text{ (when } x \gtreqless a) \ldots\ldots\ldots\ldots\ldots\ldots\ldots = \dfrac{Ma}{EI}\left(x - \dfrac{a}{2}\right)$

$\Delta_{max} \text{ (when } x = L) \ldots\ldots\ldots\ldots\ldots\ldots\ldots = \dfrac{Ma}{EI}\left(L - \dfrac{a}{2}\right)$

CONTINUITY IN STRUCTURES

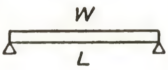
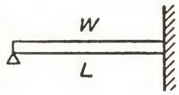
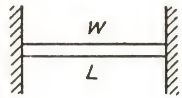
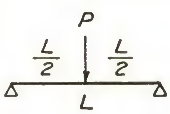
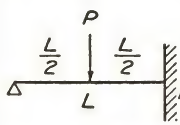
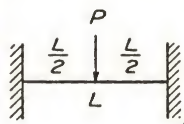
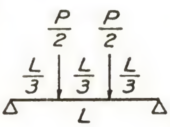
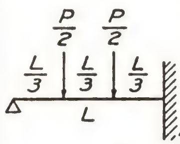
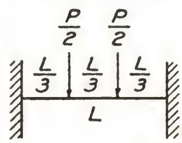
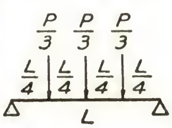
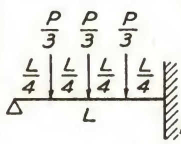
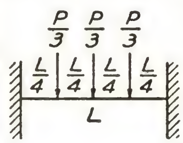
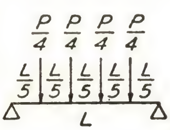
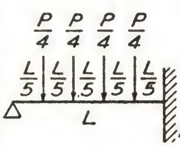
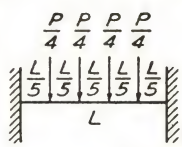
All of the tables of safe carrying capacities for flexural members given in this book are based upon loads uniformly distributed along the span length of the member; also the ratios of live to dead load (i.e., 3:1), except in a few beam tables, and of span length under consideration to the lengths of adjoining spans (i.e., longer $\lessgtr$ 1.20 shorter) are assumed to be within the limits imposed by the ACI "Building Code Requirements for Reinforced Concrete (ACI 318-56)" (701c), permitting the use of arbitrary moment coefficients that will produce reasonably safe results.

When concentrated loads are encountered, it is possible to enter the safe uniform load tables with approximately equivalent uniform loads. When span lengths or the ratio of live to dead load vary beyond Code limits, it is best to compute the moments by the Three Moment Equation or by Moment Distribution as illustrated briefly in this section and explained at length in any standard text on continuity (rigid frames).

Concentrated Loads

Sometimes, as in the case of girders carrying beams, loads are concentrated, not uniformly applied. It is possible to determine an "equivalent uniform load" that will produce the same *maximum* positive or *maximum* negative bending moment as the series of concentrated loads. These equivalent total loads, $W = wL$, are given for a number of cases in the table on page 67, where $+W$ produces same maximum positive moment and $-W$, the same maximum negative moment as do the concentrated loads. It is obvious that the shears, and the moments at other points than that where moment is maximum, will be quite different from the shears and moments produced by the concentrated loads. Hence the use of such equivalent uniform loading is not a safe way to make the final design of a flexural member. It may help in making a tentative selection for preliminary estimating purposes or in determining preliminary clearances. The table is given with this explanation as being of some guidance. Those not familiar with the general theory of bending would do well to obtain assistance in design for any cases that depart from the limitations imposed on each set of tables.

EQUIVALENT UNIFORM LOAD FOR MAXIMUM MOMENT

Single Span	End Span	Interior Span
 Max M = $\frac{WL}{8}$	 Max.−M = $-\frac{WL}{8}$ Max.+M = $+\frac{9\,WL}{128}$	 Max.−M = $-\frac{WL}{12}$ Max.+M = $+\frac{WL}{24}$
 $W = 2P$ Max.M = $\frac{PL}{4}$	 $+W = \frac{20P}{9}$ $-W = \frac{3P}{2}$ Max.−M = $-\frac{3PL}{16}$ Max.+M = $+\frac{5PL}{32}$	 $+W = 3P$ $-W = \frac{3P}{2}$ Max.−M = $-\frac{PL}{8}$ Max.+M = $+\frac{PL}{8}$
 $W = \frac{4P}{3}$ Max.M = $\frac{PL}{6}$	 $+W = \frac{128P}{81}$ $-W = \frac{4P}{3}$ Max.−M = $-\frac{PL}{6}$ Max.+M = $+\frac{PL}{9}$	 $+W = \frac{4P}{3}$ $-W = \frac{4P}{3}$ Max.−M = $-\frac{PL}{9}$ Max.+M = $+\frac{PL}{18}$
 $W = \frac{4P}{3}$ Max.M = $\frac{PL}{6}$	 $+W = \frac{34P}{27}$ $-W = \frac{5P}{4}$ Max.−M = $-\frac{5PL}{32}$ Max.+M = $+\frac{17PL}{192}$	 $+W = \frac{3P}{2}$ $-W = \frac{5P}{4}$ Max.−M = $-\frac{5PL}{48}$ Max.+M = $+\frac{3PL}{48}$
 $W = \frac{6P}{5}$ Max.M = $\frac{3PL}{20}$	 $+W = \frac{128P}{100}$ $-W = \frac{6P}{5}$ Max.−M = $-\frac{3PL}{20}$ Max.+M = $+\frac{9PL}{100}$	 $+W = \frac{6P}{5}$ $-W = \frac{6P}{5}$ Max.−M = $-\frac{PL}{10}$ Max.+M = $+\frac{PL}{20}$

+ W = total uniform load which will produce same *maximum* positive bending moment as is produced by the concentrated loads; — W, the same *maximum* negative moment.

CONTINUITY IN STRUCTURES

Solution:—Write the Three Moment Equation, omitting unnecessary terms:—

$$0 + 2M_1(L + 0.833L) + 0 = -\frac{4wL^3}{4} - \frac{w(0.833L)^3}{4}$$

Solving:—$M_1 = -\frac{(4 + 0.579)wL^2}{4 \times 3.666} = -0.3128wL^2$, which can also be written as $M_1 = -0.0782w_1L^2$.

Determine the left end reaction as $R_L = 2wL - 0.3128wL = 1.6872wL$.

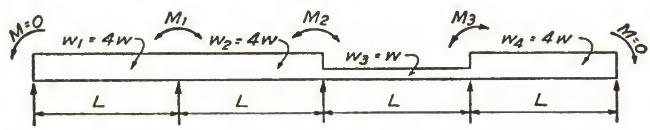
Section for zero shear and max. positive moment:—$x = \frac{1.6872wL}{4w} = 0.4218L$.

Positive moment:—$+M = \frac{R_L x}{2} = \frac{1.6872wL \times 0.4218L}{2} = 0.356wL^2$, which can also be written as $0.089w_1L^2$, or approx. $\frac{w_1L^2}{11.25}$

Observation—For only two spans, the Three Moment Equation can be written directly.* As the number of spans increases, it is necessary to work progressively across the structure, two spans at a time, resulting in a number of simultaneous equations which may become involved.

The Three Moment Equation can be used with equal facility for numerical or algebraic values; in fact algebraic values are often preferable, as they combine and cancel readily.

Example II—Given four equal spans with equal moments of inertia, no end restraint and no concentrated loads. If the live load can not exceed three times the dead load, determine the absolute maximum moment over the first interior support.



Solution:—The load arrangement for this moment to be a maximum should be as in the figure above, placing the live load on each side of the support and alternate spans each way from there.

Write the Three Moment Equations for spans 1-2, 2-3 and 3-4, simplifying each one as soon as written:—

$$0 + 2M_1(L + L) + M_2L = -\frac{4wL^3}{4} - \frac{4wL^3}{4}, \text{ so } 4M_1 + M_2 = -2wL^2 \quad (1)$$

$$M_1L + 2M_2(L + L) + M_3L = -\frac{4wL^3}{4} - \frac{wL^3}{4}, \text{ so } M_1 + 4M_2 + M_3 = -\frac{5wL^2}{4} \quad (2)$$

$$M_2L + 2M_3(L + L) + 0 = -\frac{wL^3}{4} - \frac{4wL^3}{4}, \text{ so } M_2 + 4M_3 = -\frac{5wL^2}{4} \quad (3)$$

$$4M_1 + 16M_2 + 4M_3 = -5wL^2 \quad (2)$$

$$M_2 + 4M_3 = -\frac{5wL^2}{4} \quad (3)$$

$$4M_1 + 15M_2 = -\frac{15wL^2}{4} \quad \text{(Combining (2) and (3) to eliminate } M_3) \quad (4)$$

$$60M_1 + 15M_2 = -30wL^2 \quad (1)$$

$$56M_1 = -\frac{105wL^2}{4} \quad \text{(Combining (4) and (1) to eliminate } M_2)$$

$$M_1 = -\frac{105wL^2}{224}, \text{ or } -\frac{105w_1L^2}{896}$$

---

* The equation $M_1 = -\frac{(4 + 0.579)wL^2}{4 \times 3.666}$ can be written without the need of setting down the Three Moment Equation.

CONTINUITY IN STRUCTURES

Solution:—Write the Three Moment Equation, omitting unnecessary terms:—

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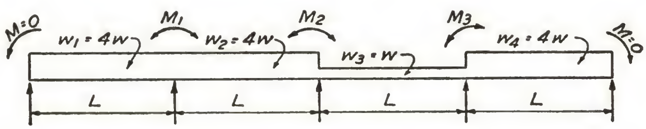

Solution:—The load arrangement for this moment to be a maximum should be as in the figure above, placing the live load on each side of the support and alternate spans each way from there.

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$$0 + 2M_1(L + L) + M_2L = -\frac{4wL^3}{4} - \frac{4wL^3}{4}, \quad \text{so } 4M_1 + M_2 = -2wL^2 \qquad (1)$$

$$M_1L + 2M_2(L + L) + M_3L = -\frac{4wL^3}{4} - \frac{wL^3}{4}, \quad \text{so } M_1 + 4M_2 + M_3 = -\frac{5wL^2}{4} \qquad (2)$$

$$M_2L + 2M_3(L + L) + 0 = -\frac{wL^3}{4} - \frac{4wL^3}{4}, \quad \text{so } M_2 + 4M_3 = -\frac{5wL^2}{4} \qquad (3)$$

$$4M_1 + 16M_2 + 4M_3 = -5wL^2 \qquad (2)$$

$$M_2 + 4M_3 = -\frac{5wL^2}{4} \qquad (3)$$

$$4M_1 + 15M_2 = -\frac{15wL^2}{4} \qquad \text{(Combining (2) and (3) to eliminate } M_3) \qquad (4)$$

$$60M_1 + 15M_2 = -30wL^2 \qquad (1)$$

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$$M_1 = -\frac{105wL^2}{224}, \quad \text{or } -\frac{105w_1L^2}{896}$$

---

* The equation $M_1 = -\frac{(4 + 0.579)wL^2}{4 \times 3.666}$ can be written without the need of setting down the Three Moment Equation.

CONTINUITY IN STRUCTURES

Observation—The continued application of the Theorem is here illustrated, and the simplicity of application to algebraic values shown.

These examples show how the Three Moment Equation can be used to solve for bending moments in continuous runs of beams. They could be extended indefinitely, as the theory of continuity is a subject in itself. If the moment over the first support is not zero, but is known by reason of a cantilevered end which produces a definite moment, that value (usually negative) can be substituted in the Three Moment Equation instead of zero.

The user is counselled to familiarize himself with this Equation and use it for cases that are outside the scope of these tables.

Moment Distribution

Moment Distribution is essentially a method of successive approximations and, when applied to continuous flexural members in a single line, is best understood as step (a) plus the repeated applications of steps (b) and (c) below:—

(a) Assume that all joints (supports) of the loaded run of beams are fixed, without rotation. The first step, then, is this:—

I—At each end of each span, compute the fixed end moment (F.E.M.) due to the loading of that span with joints assumed fixed.*

(b) For static equilibrium, the moment at the right end of the left span must equal that at the left end of the abutting right span, i.e., the moments acting on the joint must balance. The assumed fixed-end-moments will rarely be numerically equal and the second step is to relieve or "unlock" each of the joints, one at a time, and permit rotation until the moments on either side balance each other. The two meeting beams become resisting levers to take up the unbalanced moment in any joint, and will divide it in proportion to their relative stiffnesses as measured by the ratio I/L. This step is known as "distribution."

II—Having previously computed the relative stiffnesses ($K = I/L$) of the two beams, "distribute" the unbalanced moment at their junction in proportion to their stiffness.

(c) Since the adjoining beams described as resisting levers are considered artificially fixed at their far ends, it is impossible for them to receive a moment change at one end without inducing a moment at the other end, whose magnitude is a function of the shape of the beam. For a prismatic beam, this "carry-over" moment is one-half of the moment at the opposite end.

III—Carry over a portion (one-half for straight, prismatic beams) of the distributed moment to the opposite end of each beam.

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* Values for a number of loading conditions are given in the tables on pages 72-75.

CONTINUITY IN STRUCTURES

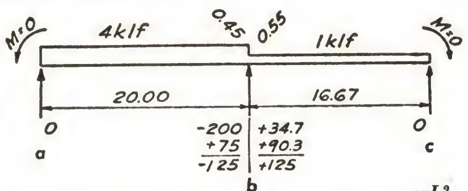
(d) Because step III was performed subsequently to and independently of step II, the induced moment in step III unbalances what was previously a balanced condition at the joint where the carry-over moment was added. Hence it is necessary to repeat steps II and III as often as desired to obtain closer and closer approximations of the true values. The steps can be stopped at any time, but only after a distribution and before a carry-over.

IV—Repeat steps II and III for closer approximations, stopping after a distribution of unbalanced moments.

Signs—It is customary to consider the moment applied to a supporting joint by a loaded beam as positive when it tends to cause clockwise rotation of the joint. (Thus, in the figure below, the F.E.M. at the right end of beam *ab*, usually designated as M_{ba}, is negative; that at the left end of beam *bc*, M_{bc}, is positive. The sign of the carry-over moment is the same as that of the distributed moment which causes it.

One variation of this procedure must be explained before illustrating with examples. In the case of a freely supported end, it is too tedious to assume that end fully fixed and then work it back to a free end by successive approximations, though that is quite possible. Less computing is done if the stiffness coefficient of an end (freely-supported) span is taken as $\frac{3}{4}I/L$, and if the moment at the free end is assumed zero (as it must be) and at the interior end as that of a beam supported on one end and fixed on the other $\left(-\frac{wL^2}{8}\right.$ for a uniform load).

Example—To verify the derivation in Example I, page 68, assume $w_1 = 4$ klf, $w_2 = 1$ klf, $L = 20$ ft, and $0.833L = 16.67$ ft, and compute the bending moment over the support by moment distribution.



Solution:—Write the end moments in each span:—0 and $\frac{wL^2}{8} = -200$ kf for the left span, and $\frac{wL^2}{8} = +34.7$ kf for the right span. Determine the stiffnesses (I/L ratios) of the two spans. Since they have the same moment of inertia and similar end conditions and are each prismatic, their stiffnesses vary inversely as the spans, 1.0 for the left span and 1.2 for the right, so that the left span takes $\frac{1.0}{2.2}$, or 45 per cent of any unbalanced moment, and the right span takes $\frac{1.2}{2.2}$, or 55 per cent (written on the diagonal in the figure above).

The unbalanced moment is $-200 + 34.7 = -165.3$, so $+165.3$ is needed to resist this, of which 45 per cent, or 75.0, is provided by the beam in the left span and 55 per cent, or 90.3, by the beam in the right span. Because the beams were adjusted to a free outer end before starting, there is no carry-over and the work is complete. Adding the terms on each side of the support results in a negative moment of 125 ft-kips, in balance on either side of the support, and this compares with the value from Example I, page 68, which was $M_1 = -0.0782w_1L^2 = 125.1$ ft-kips.

COEFFICIENTS FOR MOMENTS IN BEAMS WITH FIXED ENDS (F.E.M.)

Moments In Beams Of Constant Section And With Fixed Ends

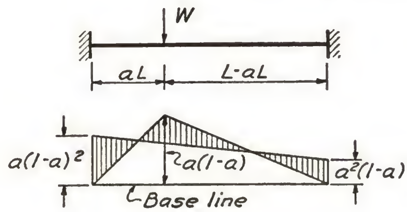
$M = m \times W \times L$

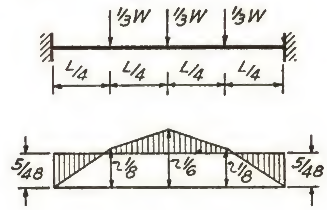
m = coefficient taken from diagram

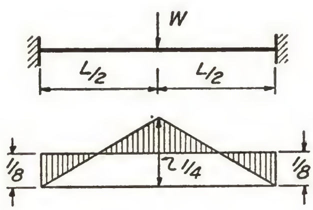
W = total load on beam

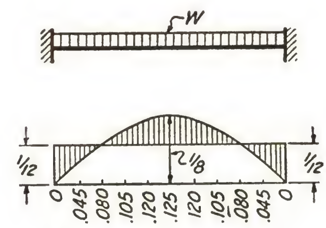
L = length of beam

a = length in terms of L

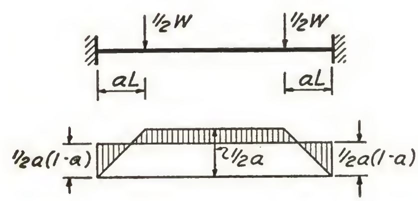


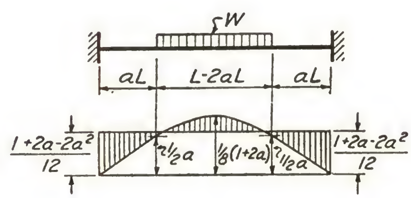


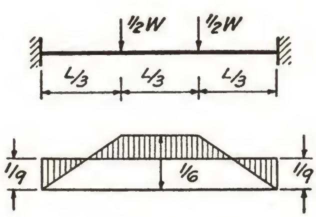


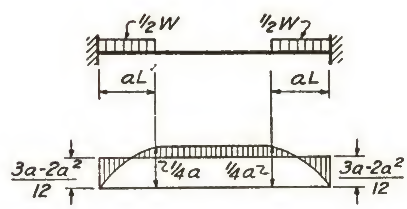


(Values at 1/10 points)

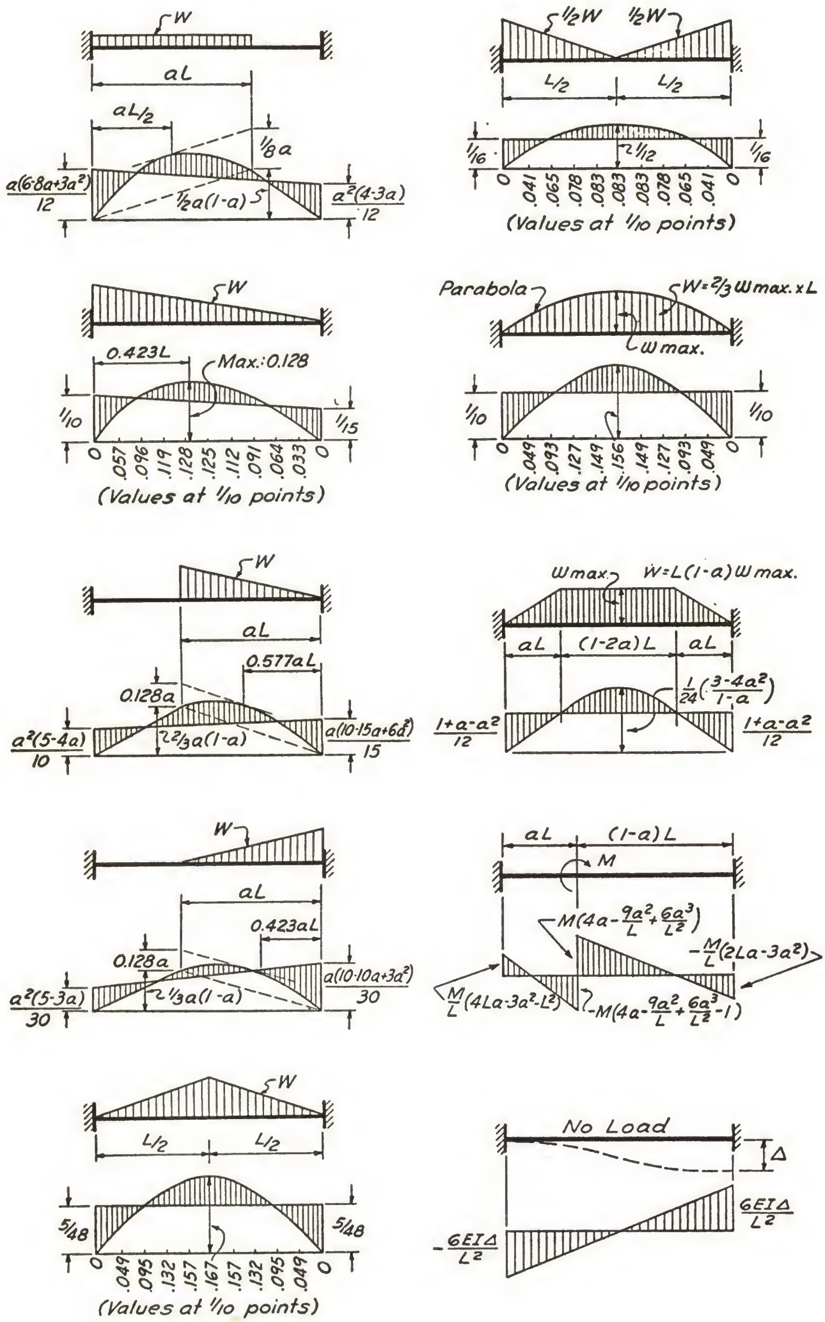








COEFFICIENTS FOR MOMENTS IN BEAMS WITH FIXED ENDS (F.E.M.)

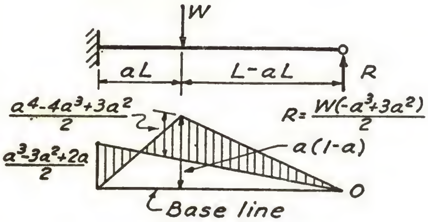


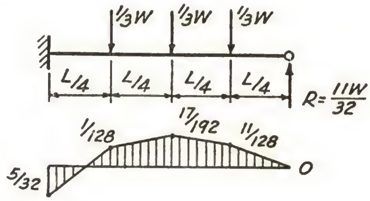
MOMENTS IN BEAMS OF CONSTANT CROSS-SECTION—ONE END FIXED

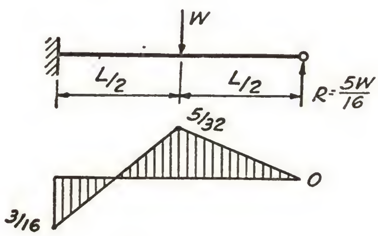
Moments In Beams Of Constant Section - One End Fixed, One End Free

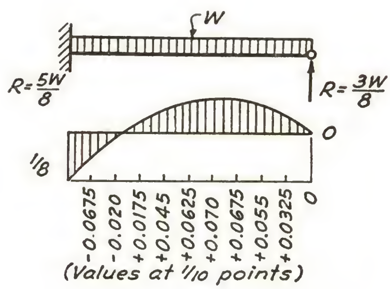
$M = m \times W \times L$

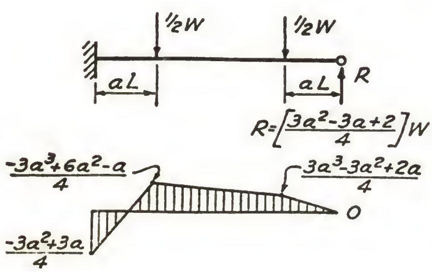
m = coefficient taken from diagram
W = total load on beam
L = length of beam
a = length in terms of L

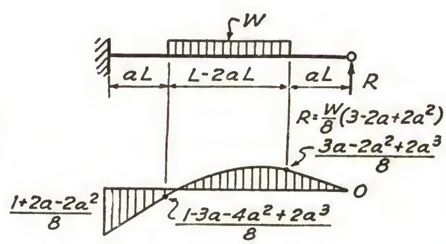


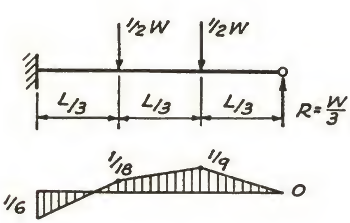


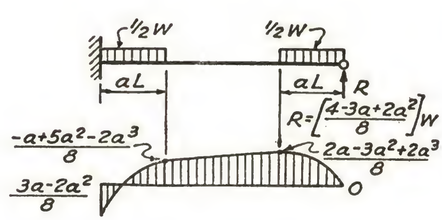




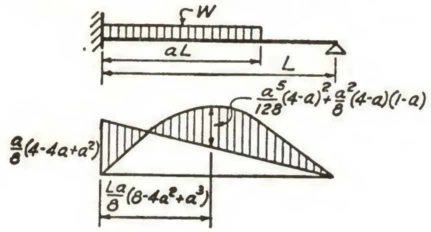


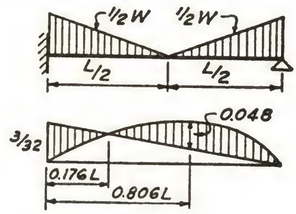


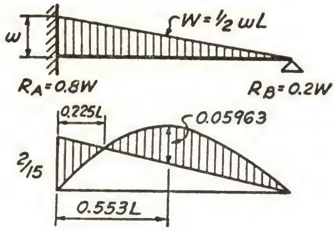


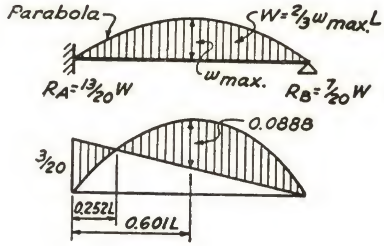


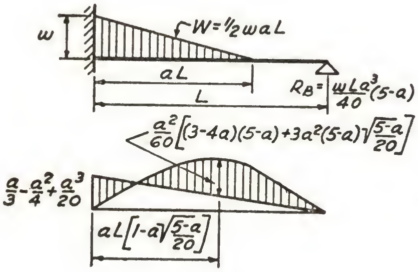
MOMENTS IN BEAMS OF CONSTANT CROSS-SECTION—ONE END FIXED

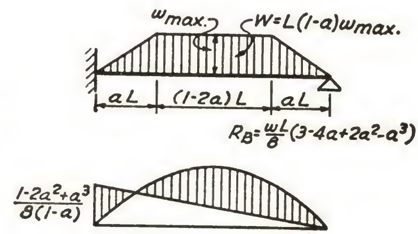


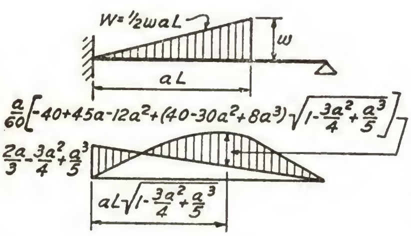


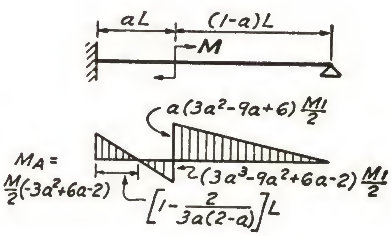


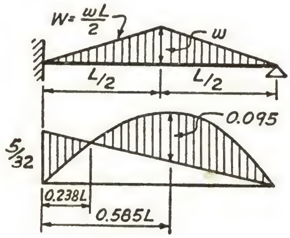


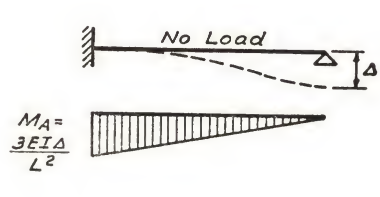










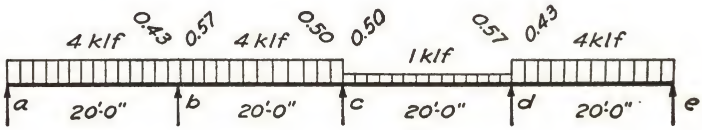


CONTINUITY IN STRUCTURES

Observation—With moment distribution it is almost impossible to work with algebraic terms, but the successive steps make the use of numerical values quite simple. A clear mental picture of each step is possible, such as *moment at the free end, fixed end moment, stiffness of members, participation in unbalanced moment, distribution of moment,* and *carry-over*. Quick, practical results are available to the designer, carried to such degree of precision as he desires.

Example—To verify the value of negative moment over the first interior support for Example II, page 69, assume $w_1 = w_2 = w_4 = 4$ klf, $w_3 = 1$ klf, and $L = 20$ ft, and determine the bending moment over the first interior support.

Solution:—The moments (kf) at the outer ends as given on line (1), figure below, are 0; at the other supports in order, $\frac{wL^2}{8} = -200$; $\frac{wL^2}{12} = +133.3$, -133.3, $+33.3$, -33.3; and $\frac{wL^2}{8} = +200$. The stiffnesses of the four spans, taking the free-end spans as $\frac{3}{4}I/L$, are 0.75, 1.00, 1.00 and 0.75. The distribution factors at each joint work out:—0, $\frac{0.75}{1.75} = 0.43$, $\frac{1.00}{1.75} = 0.57$, $\frac{1.00}{2.00} = 0.50$, 0.50 and, again, 0.57, 0.43, 0.



	ab	ba	bc	cb	cd	dc	de	ed	
(1)	0	-200	+133.3	-133.3	+33.3	-33.3	+200	0	FEM
(2)	0	+28.6	+38.1	+50.0	+50.0	-95.4	-71.3	0	Dist.
(3)	—	(-171.4)	(+171.4)	(-83.3)	(+83.3)	(-128.7)	(+128.7)	—	Σ
(4)	0	0	+25.0	+19.1	-47.7	+25.0	0	0	C.O.
(5)	0	-10.7	-14.3	+14.3	+14.3	-14.3	-10.7	0	Dist.
(6)	—	(-182.1)	(+182.1)	(-49.9)	(+49.9)	(-118.0)	(+118.0)	—	Σ
(7)	0	0	+7.2	-7.2	-7.2	+7.2	0	0	C.O.
(8)	0	-3.1	-4.1	+7.2	+7.2	-4.1	-3.1	0	Dist.
(9)	0	-185.2	+185.2	-49.9	+49.9	-114.9	+114.9	0	Σ

The unbalanced moment over the first interior support of -66.7 is "distributed" as 28.6 and 38.1 on line (2); and moments at other supports as indicated on the same line of figures. Now all of the joints are "in balance," and a fair approximation of a result can be had from the next line of figures enclosed in parentheses on line (3). However, the individual beams are not in equilibrium because of the induced moments on their other ends, and it is necessary to "carry over" one-half of these moments as shown below the parentheses on line (4).

Repeating the "distribution" of these smaller, unbalanced moments gives the values on line (5). Should work be stopped at this point, the final values would be as shown in parentheses on line (6).

For comparison, compute the value from the result of Example II, page 69, as $M_1 = -\frac{105 w_1 L^2}{896} = 187.5$ ft-kips. Thus it is seen that, with two cycles of distribution, a fairly good approximation is developing. Lines (7) and (8) repeat another cycle. It is felt that, in view of the many assumptions inherent in continuous structures, the value of 185.2 ft-kips (Line 9) is closely enough in agreement with the 187.5 ft-kips from Example II. The user may go through other cycles of distribution and carry-over to see how the value gradually approaches the theoretical one.

The summations here shown in parentheses are not recorded in practical computations and are included here to illustrate the increasing precision, cycle by cycle.

CONTINUITY IN STRUCTURES

Observation—Fairly reliable values can be obtained after a couple of cycles. The carry-overs show the degree of precision in each cycle and the work may be stopped when these CO values seem negligible. The computer has a clear picture of the action of the structure and the meaning of each step as he goes along—in fact, if he does not have such a clear picture, he should stop and get it.

For design purposes, it is usually sufficient to determine with reasonable accuracy the moment at a certain point for a maximum loading condition. Many methods have been proposed for shortening the computations. (As always, understand the method before trying simplifications.) For a workable result the effect of a load in the first span only of the figure on page 76 can be approximated as D_{bc} ($FEM_{ba} - \frac{1}{2}D_{ab}\ FEM_{ab}$) and of a load in the second span only as $D_{ba}(-FEM_{bc} + \frac{1}{2}D_{cb}\ FEM_{cb})$, where D is the distribution factor. Applied to this problem:

$$\text{1st Span } 0.57(-200 - 0 \times 0) = -114.0\text{ kf}$$

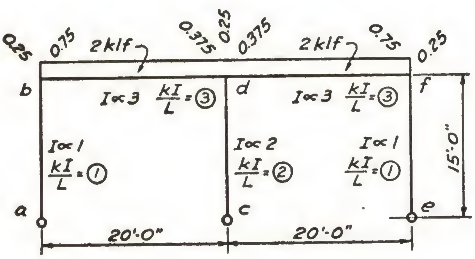
$$\text{2nd Span } 0.43\left(-133.3 - \frac{0.50 \times 133.3}{2}\right) = -\ 71.6$$

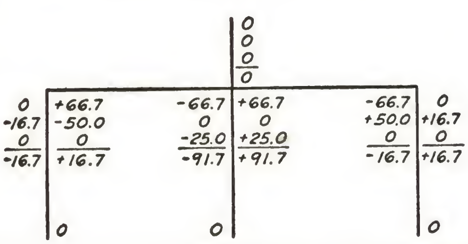
$$\overline{-185.6\text{ kf}}$$

The examples shown are not complicated and present the Three Moment Equation as a simpler device than it works out in practice. Moment Distribution can be extended to take care of columns as well as beams, and to take care of members with variable moments of inertia (such as haunched beams and arched beams of various shapes). Each is a helpful tool in its place and each should be thoroughly understood by a designer of concrete structures.

To illustrate the application of Moment Distribution to a simple bent which is symmetrical in its framing and symmetrical in its loading (thus preventing sidesway or lurch), the following example is given. For more complicated frames, for frames with unsymmetrical loading or unsymmetrical members, for frames with varying moments of inertia or for frames undergoing horizontal loads, consult any good text on Moment Distribution.

Example—For the three-legged symmetrical bent illustrated in the figure below with pin-connected column bases and a symmetrical load of 2 kips per running foot, the moments of inertia of the columns being as 1 and 2 and that of the beams as 3, compute the negative moments at the tops of the three columns.





Solution:—Since the columns are pin-connected at the base, take $\frac{3}{4}\frac{I}{L}$, but for the beams use $\frac{I}{L}$ to obtain the following:—

$$\frac{3}{4}\frac{I_{ab}}{L} = \frac{3}{4}\frac{I_{ef}}{L} = \frac{3}{4}\frac{1}{15} = 0.05, \text{ varies as } 1$$

$$\frac{3}{4}\frac{I_{cd}}{L} = \frac{3}{4}\frac{2}{15} = 0.10, \text{ varies as } 2$$

$$\frac{I_{bd}}{L} = \frac{I_{df}}{L} = \frac{3}{20} = 0.15, \text{ varies as } 3$$

The distribution at joints b and f will then be $\frac{1}{1+3}$, or 0.25, in the column and $\frac{3}{1+3}$, or 0.75, in the beam. At joint d the distribution to the column will be $\frac{2}{3+2+3}$, or 0.25; and to each

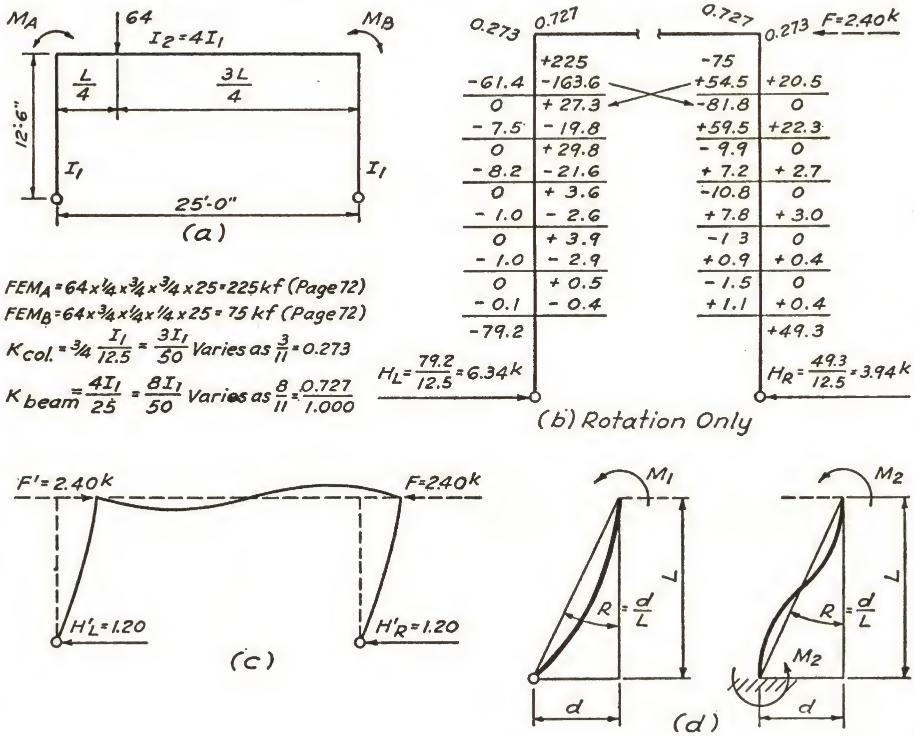
beam, $\frac{3}{3+2+3}$, or 0.375.

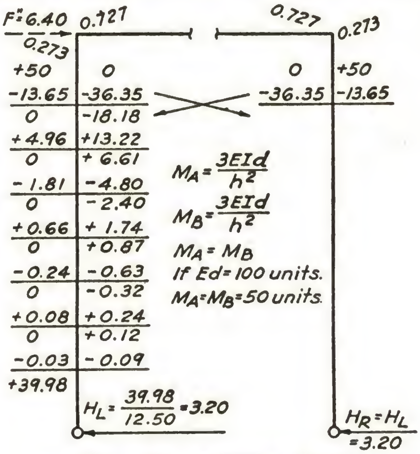
In the figure on page 77, the *FEM*'s are computed as $\frac{2 \times 20 \times 20}{12} = 66.7$. One distribution is made as noted and one carry-over. The results are totalled to give the moments desired. Note that the figures outside of the outer legs represent the column moments; those inside the outer legs, the beam moments. At joint *d* the two lower computations represent the beam moments and the upper one the column moment.

Sidesway

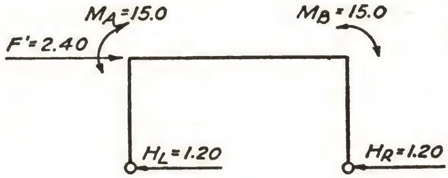
A structure unsymmetrical in its framing or unsymmetrical in its loading will be subjected to sidesway or lurch, so joint moments computed for rotation only must be corrected for translation. Often the simplest procedure is to solve first for rotation, see what horizontal force would be required to prevent sidesway, and then correct the values for the effect of a negative force of this amount.

Example I—For a two-legged symmetrical framed bent with hinged columns and unsymmetrical load as shown in (a), determine the corner moments M_A and M_B.





(e) Translation Only



$$M_A = M_B = \frac{2.40}{6.40} \times 39.98 = 15.0$$

$$H_L = H_R = \frac{2.40}{6.40} \times 3.20 = 1.20$$

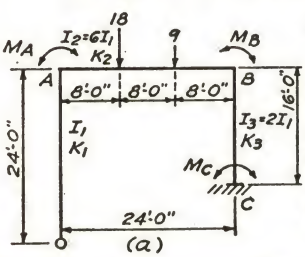
(f)

$$M_A = -79.2 + 15.0 = -64.2^{kf}$$
$$M_B = +49.3 + 15.0 = +64.3^{kf}$$
$$H_L = 6.34 - 1.20 = 5.14^{k}$$
$$H_R = 3.94 + 1.20 = 5.14^{k}$$

(g) Combined

The fixed end moments in the beam and the stiffness ratios are first determined, as on (a). Then in (b) a moment distribution is made, resulting in corner moments of -79.2 and $+49.3$ *kf*, respectively. From these, the horizontal reactions at the bottoms of the columns are computed as 6.34 and 3.94, respectively. Since these are not equal to each other, a horizontal force ($F = 2.40$ kips), shown dotted on (*b*), would be required to restore equilibrium. This means that while the bent would deflect as shown by heavy lines in (c), the distribution has assumed the presence of a nonexistent force $F = 2.40$ kips, holding the two upper corners directly over the bases of the columns, so it is necessary to add the effects of a force $F' = 2.40$ kips in the opposite direction, i.e., determine the corner moments that it develops and combine them with those obtained in (b). While this is relatively simple for this bent, a procedure will be followed in (e) that can be applied to more complicated bents. As shown in (d), a member free on one end and fixed at the other, if offset a certain distance, *d*, develops a moment at the attached end of $M_1 = \dfrac{3EIR}{L} = \dfrac{3EId}{L^2}$, while one fixed at both ends develops a moment $M_2 = \dfrac{6EIR}{L} = \dfrac{6EId}{L^2}$. To determine the effects of a horizontal force, in (e), for equal horizontal offsets at the top of each column (i.e., no change in length of the connecting beam), express the corner moments at the tops of the columns in terms of such offset (for example, assume here a value for *Ed* of, say, 100 units); distribute the moments and compute the horizontal forces to find $F'' = 6.40$ kips; then, as in (f), the moments and shears for a force $F' = 2.40$ kips will be $\dfrac{2.40}{6.40}$ of those shown on (e). Then (b) and (f) are combined for the final moments, as tabulated in (g). For such a symmetrical case, this second distribution could be done mentally by making $H_L = H_R = F'/2 = 1.20$, and $M_A = M_B = 1.20 \times 12.5 = 15.0$, but the full explanation is given for convenience in discussing the next example.

Example II—For a two-legged unsymmetrically framed bent with unsymmetrical loads as shown in (a) in the figure, determine the corner moments M_A, M_B and M_C.



(a)

$K_1 = \frac{3}{4}\frac{I_1}{24} = \frac{I_1}{32}$ Varies as 1 $\quad \frac{1}{9}$

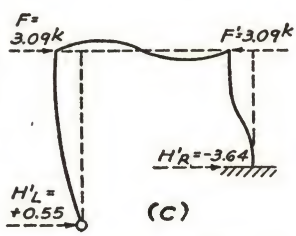
$K_2 = \frac{6I_1}{24} = \frac{8I_1}{32}$ Varies as 8 $\quad \frac{8}{9} \quad \frac{8}{12} = \frac{2}{3}$

$K_3 = \frac{2I_1}{16} = \frac{4I_1}{32}$ Varies as 4 $\quad \frac{4}{12} = \frac{1}{3}$

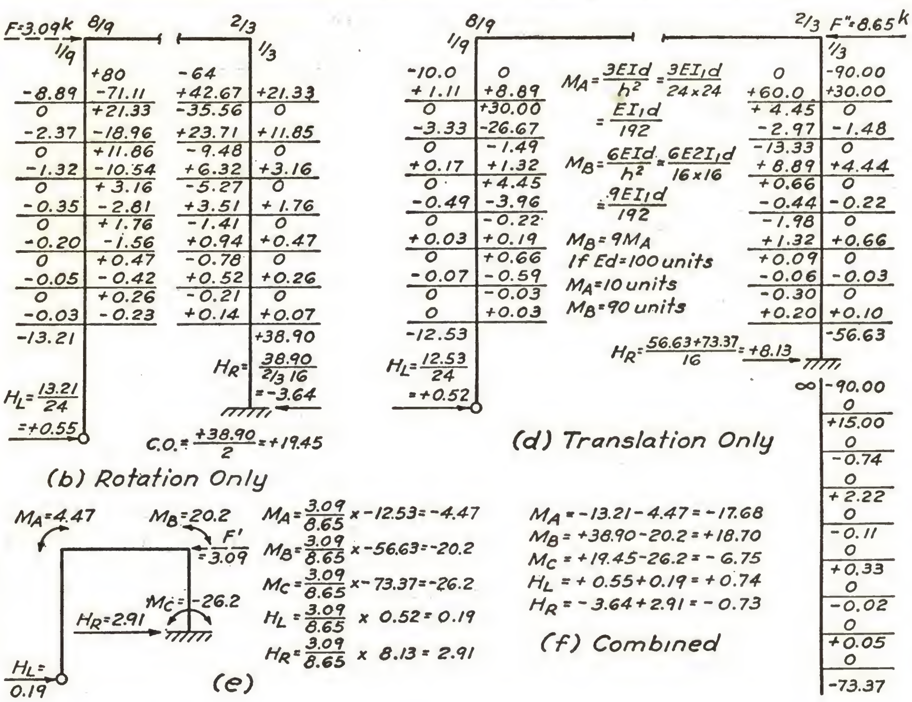
Using formula from page 72:-

$FEM_A = 18 \times \frac{1}{3} \times \frac{2}{3} \times \frac{2}{3} \times 24 = 64$ kf
$9 \times \frac{2}{3} \times \frac{1}{3} \times \frac{1}{3} \times 24 = 16$
80 kf

$FEM_B = 18 \times \frac{2}{3} \times \frac{1}{3} \times \frac{1}{3} \times 24 = 32$ kf
$9 \times \frac{1}{3} \times \frac{2}{3} \times \frac{2}{3} \times 24 = 32$
64 kf



(c)



The stiffness ratios and the fixed end moments in the beam are first determined in (a). Then a moment distribution is made in (b), resulting in corner moments at the tops of the columns of -13.21 and $+38.90$ *kf*, respectively. Then the horizontal reactions at the feet of the columns are computed, realizing that the carry-over moment at the foot of the fixed base column will be exactly half the moment at the top of the same column, and, therefore, the horizontal reaction is obtained by dividing the corner moment by two-thirds the height of the column. This gives reactions of 0.55 and 3.64, and since these are not equal to each other, a horizontal force of $F = 3.09$ kips would be required to restore equilibrium, as shown dotted in (b). Again, this means that while the bent would deflect as indicated by the heavy lines in (c), the distribution has assumed the presence of a nonexistent force of $F = 3.09$ kips, holding the two upper corners directly over the bases of the columns. It becomes necessary to add the effects of a force $F' = 3.09$ kips in the opposite direction, i.e., determine the corner moments induced by this lateral force and combine them with those obtained in (b). As shown in the preceding example, the moments at the tops of the two columns for the same horizontal offset are in the ratio 1:9 as computed on (d). Assuming an indefinite displacement represented by $Ed = 100$ units would induce corner moments of 10 and 90 *kf*, respectively. A distribution gives corner moments at the tops of the columns of -12.53 and -56.63 and at the bottom of the fixed base column of -73.37 with horizontal reactions of $+0.52$ and $+8.13$, requiring a horizontal force $F'' = 8.65$ kips for equilibrium. Since 3.09 is all that is required, all values are reduced in the proportion 3.09/8.65 and recorded on (e). Finally, at (f), the values are combined for the final moments and horizontal reactions.

Observation—It is not possible to go much further into moment distribution in a handbook. These examples merely illustrate the possibilities of applying the method to framed bents.

STIFFNESS FACTOR

In designing by Moment Distribution (pp. 70-80), the unbalanced moment is "distributed" to the meeting beams in proportion to their stiffness and the degree of restraint of the far end, $CK = CI/L$. For far end fixed, $C = 4$; for far end simply supported, $C = 3$. When the far ends are fixed, $C = 4$ at all times and may be omitted from the calculation. When a structure is symmetrical both in itself and as to load, work with one-half of the structure, noting that $C = 2$ if an equal but reversed moment occurs at the far end of the central span, $C = 6$ if the far end moment is equal and in the same direction.

For computing stiffness, the length of the member is ordinarily taken as the distance from center to center of its supports, rather than the clear span.

The moment of inertia of the cross section can be obtained in one of two ways:—

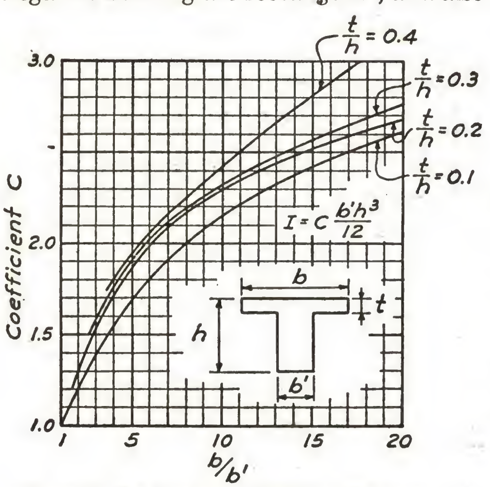
(a) For preliminary computations, use the moment of inertia of the gross outline of the concrete section, omitting the reinforcing steel. If the same method is applied to all of the members, this is fairly reasonable. In an ordinary continuous tee-beam, there is a tee section near midspan and a rectangular section at either end, and the point of inflection moves under different loading conditions, so a high degree of precision is not obtainable.

(b) Some designers, feeling that a heavily reinforced beam should have more stiffness than a lightly reinforced one, take the moment of inertia of the transformed area, though there is a question whether this is any more precise, and it can be done only after considerable preliminary designing to establish all the needed data.

For the "I" of a rectangular section reinforced on both faces, good illustrations are shown in the examples on pages 276, 278, 297, and 335.

For a rectangular section with reinforcement on one side only, "I" would be computed in the same manner except that certain terms would drop out.

The moment of inertia of a tee-beam is difficult to appraise properly, because, being an integral part of a floor system, there is a question of how much slab should be included as a tee, e.g., the same as recommended for stress computations (ACI 318-56 705); also because the portions of the beam undergoing negative bending are rectangular; and also because of the bending up of longitudinal bars. The designer may compute "I" either for a homogeneous concrete tee section or for the transformed area. Since the stiffnesses must be available before frame analysis can be started and the design can not be completed without assuming stiffnesses, it is best, in preliminary work and work where extreme precision can not be expected, to take the "I" of a tee-beam as about 2 to 2.25 times that of the stem only. The chart can be used for a slightly more precise value.

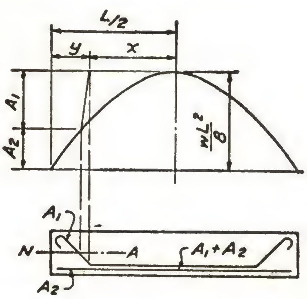


MOMENT OF INERTIA OF TEE BEAMS

BENDING OF BEAM BARS

Often for simplicity in placing, some of the tension bars in beams, slabs or joists are bent or "trussed" from the bottom in the central part of a span to the top at either support, and extended into the adjoining spans. Enough steel must be in the bottom at all points to provide resisting moment at least equal to the positive bending moment at that point under any position of the assumed load, and enough must be in the top to take care of the negative bending moment except that additional loose top bars can be added to make up any deficiency.

In a **single span** simply supported and uniformly loaded, the moment curve is parabolic (figure below) and bottom steel can be bent up just outside of the moment curve as shown in the figure and computed as follows (L being the span):—



$$x^2 : \left(\frac{L}{2}\right)^2 = \frac{A_1}{A_1 + A_2}$$

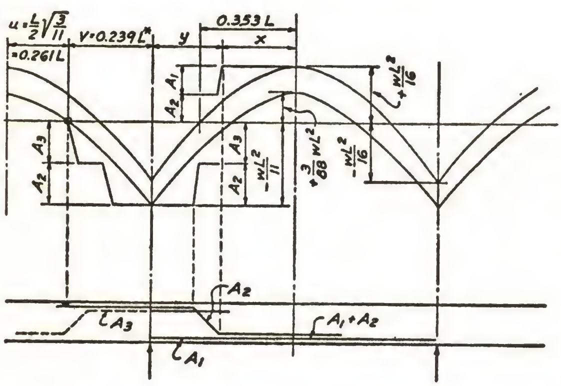
For determining bend-up point of any percentage of tension bars:

$$x = \frac{L}{2}\sqrt{\frac{A_1}{A_1 + A_2}}$$

If $A_1 = A_2$, $x = 0.353L$

$y = 0.147L = L/7$ closely.

For **interior spans** of continuous beams of approximately equal spans and uniformly loaded, ACI 701c establishes coefficients for positive and negative moment that give moment curves about as shown in the figure below, making it possible to compute bending points about as follows (L being the span):—



$$x^2 : (0.353L)^2 = \frac{A_1}{A_1 + A_2}$$

For determining bend-up point of any percentage of tension bars:

$$x = 0.353L\sqrt{\frac{A_1}{A_1 + A_2}}$$

If $A_1 = A_2$, $x = 0.250L$

$y = 0.250L = L/4$

For the extension of truss bars into the adjoining span, if $A_3 = A_2$, each being proportional to $\frac{1}{2}\frac{wL^2}{11}$, $u = \frac{L}{2}\sqrt{\frac{3}{11}} = 0.261L$, and $v = 0.239L = L/4$ closely.*

For **end spans** of continuous beams uniformly loaded, the moment parabolas are no longer symmetrical. If l is the span and

* ACI 902(a) requires that one-third of the bars be carried beyond this point of inflection $1/16L$, $d/2$, or sufficient to develop by bond one-half the stress in the bars.

BENDING OF BEAM BARS

$M = wl^2/11$, zero shear is $l\sqrt{2/11}$, or $0.426l$ from the free end. Then, $x = 0.426l\sqrt{\dfrac{A_1}{A_1 + A_2}}$. If $A_1 = A_2$, $y = 0.125l$, or $l/8$ from the free end; and from the continuous end $y \lessgtr l\,(1.0 - 0.426 - 0.301) = 0.273l$, or $3/11l$, slightly greater than $l/4$. Because of the many variables, y is often made $l/7$ from the free end and $l/5$ from the continuous end, leaving $0.657l$ horizontal near the middle of the span.

When A_1 and A_2 are not approximately equal or when the loads or spans vary considerably, values can be computed as shown or scaled from a fairly accurately sketched set of moment curves.

The curves on page 84 are helpful. For a single span, the percentage, $\dfrac{A_1}{A_1 + A_2}$, can be read downwards from the vertex, and the location of the bend-up points in percentages of l are horizontally opposite. For end spans, the upper curve is drawn for $M = +wl^2/11$ and the lower one for $M = -wl^2/10$, so that all the bend-up and bend-down points are read in percentages of l. For continuous spans, the curves are drawn for $M = +wl^2/16$ and $M = -wl^2/11$, respectively.

Example I—If a beam on a single span of 20 ft is reinforced with 6-#8 bars, 3 straight and 3 trussed, how should they be bent up?

Solution—Refer to upper diagram on page 84. Bend the first bar according to the ratio obtained by dividing the number of bent bars between center of span and point in question to the total steel area, in this case 1:6 = 1/6 = 0.167, so $y = 0.30l$ from either end. The second bar is bent up for 2/6 or 0.333, where $y = 0.21l$. The third bar is bent for 3/6 or 0.50, where $y = 0.147l$.

Example II—If the beam in Example I were the end span of a continuous beam, how should the bars be bent up? Assume top steel consists of 3-#8 bars bent up in this span plus 3-#8 bars from adjoining span plus 1-#7 added straight top bar and determine bend-down points.

Solution—Refer to center diagram on page 84, using $wl^2/11$ curve for maximum positive moment. Bend up first bar (1/6 = 0.167) at $y = 0.26l$ and $0.60l$ from free end; second bar (2/6 = 0.333) at $y = 0.17l$ and $0.68l$ from free end; and third bar (3/6 = 0.50) at $y = 0.12l$ and $0.73l$ from free end.

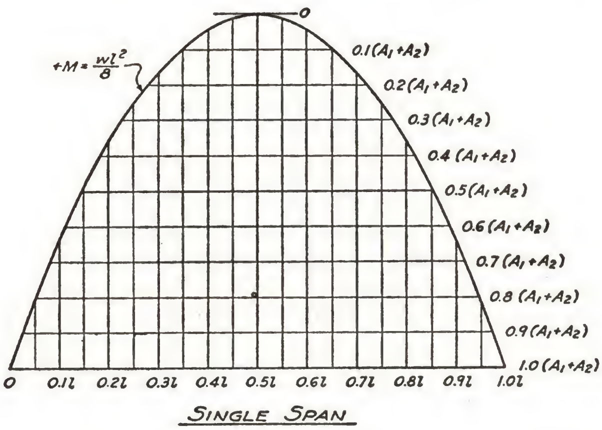
For bending down bars, use the negative moment curve, $-wl^2/10$, and, since the bars differ in size, proportion by areas instead of number of bars. The #7 top bar is 0.60/5.34 = $0.113A_s$ and must extend $0.02l$ = 0.4 ft beyond the face of the support, but at least 17 bar diameters = 1.25 ft for bond, and as much more as required for practical placement. The three truss bars represent, respectively, 1.39/5.34 = $0.26A_s$, 2.18/5.34 = $0.408A_s$ and 2.97/5.34 = $0.557A_s$ and cannot bend down within $0.045l$ = 0.9 ft, $0.075l$ = 1.5 ft, and $0.10l$ = 2.0 ft of the face of the support, each of which can be increased as necessary to produce a 45° slope from the corresponding bend-up point. The bars from the adjacent span represent 3.76/5.34 = $0.705A_s$, 4.55/5.34 = $0.853A_s$, and 5.34/5.34 = $1.0A_s$ and must extend at least $0.13l$ = 2.6 ft, $0.17l$ = 3.4 ft, and $0.20l$ = 4.0 ft into this span, plus at least 12 diameters = 1.0 ft for anchorage. Often all three bars would be carried to $l/4$ = 5.0 ft, or even a little beyond that to be sure of covering the entire negative zone.

The tables on page 85 are helpful in detailing bent bars in terms of the clear span l.

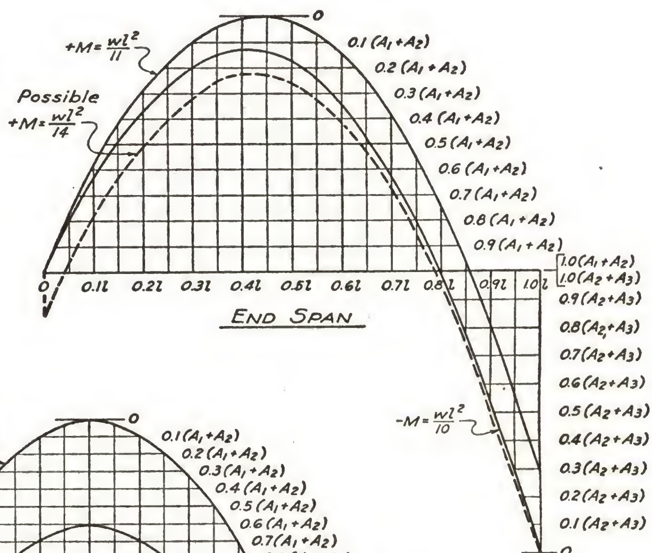
BENDING OF BEAM BARS

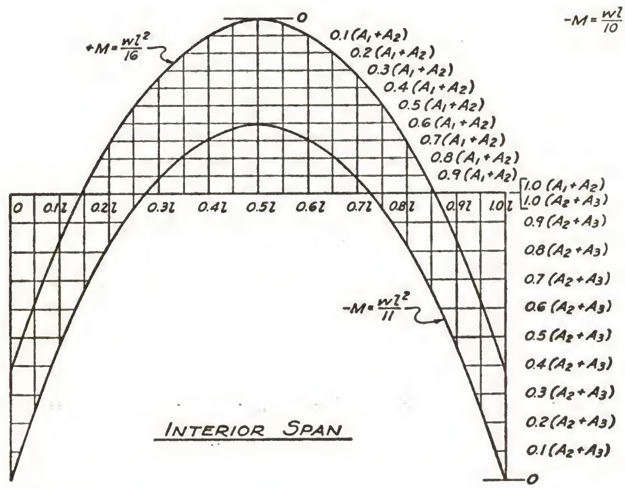
DIAGRAMS FOR DETERMINING BEND POINTS

For explanation see pages 82-83.

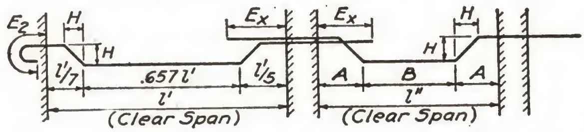


Use dotted curve for designs with end restraint, $+M=\frac{wl^2}{14}$.



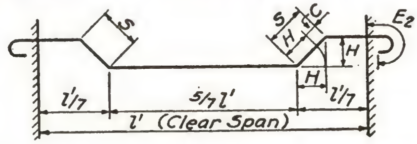


BENDING OF TRUSS BARS



E_2 = 17 bar diameters (24 if 12″ of concrete below bars), straight if possible, bent if necessary.

For slant, S, and increment, $I = 2C$, see page 14.



E_x = not less than $\frac{l''}{16}$ for span l' $\left(\frac{l'}{16}\text{ for span } l''\right)$, or d, or half-bond length past point of inflection, sometimes obtained by extending half the bars to $\frac{l'}{3}$ or $\frac{l''}{3}$, whichever is greater, and balance to $\frac{l'}{6}\left(\frac{l''}{6}\right)$, and sometimes obtained by extending all bars to $\frac{l'}{4}$ or $\frac{l''}{4}$, or 17 bar diameters past bend-down point (24 bar diameters if 12″ of concrete below bars).

A is frequently taken as $\frac{l''}{5}$ for slabs and joists, $\frac{l''}{4}$ for beams, then

B is $\frac{3l''}{5}$ for slabs and joists, $\frac{l''}{2}$ for beams

l' = Span	PERCENTAGES OF SPAN LENGTH										
	$l'/16$	$l/7$	$0.15l'$	$l'/6 = 0.167l'$	$l'/5 = 0.20l'$	$l'/4$	$l'/3 = 0.33l'$	$l' - l'/4 - l'/4 = l'/2$	$l' - l'/5 - l'/5 = 0.60l'$	$l' - l'/7 - l'/4 = 0.657l'$	$l' - l'/7 - l'/7 = 5/7l'$
8′-0	0′-6	1′-2	1′-2	1′-4	1′-7	2′-0	2′-8	4′-0	4′-10	5′-3	5′-9
8′-6	0′-6	1′-3	1′-3	1′-5	1′-9	2′-2	2′-10	4′-3	5′-1	5′-7	6′-1
9′-0	0′-7	1′-4	1′-4	1′-6	1′-10	2′-3	3′-0	4′-6	5′-5	5′-11	6′-5
9′-6	0′-7	1′-5	1′-5	1′-7	1′-11	2′-5	3′-2	4′-9	5′-8	6′-3	6′-9
10′-0	0′-8	1′-5	1′-6	1′-8	2′-0	2′-6	3′-4	5′-0	6′-0	6′-7	7′-2
10′-6	0′-8	1′-6	1′-7	1′-9	2′-1	2′-8	3′-6	5′-3	6′-4	6′-11	7′-6
11′-0	0′-8	1′-7	1′-8	1′-10	2′-2	2′-9	3′-8	5′-6	6′-7	7′-3	7′-10
11′-6	0′-9	1′-8	1′-9	1′-11	2′-4	2′-11	3′-10	5′-9	6′-11	7′-7	8′-3
12′-0	0′-9	1′-9	1′-10	2′-0	2′-5	3′-0	4′-0	6′-0	7′-2	7′-11	8′-7
12′-6	0′-9	1′-10	1′-11	2′-1	2′-6	3′-2	4′-2	6′-3	7′-6	8′-2	8′-11
13′-0	0′-10	1′-11	1′-11	2′-2	2′-7	3′-3	4′-3	6′-6	7′-10	8′-6	9′-3
13′-6	0′-10	1′-11	2′-0	2′-3	2′-8	3′-5	4′-5	6′-9	8′-1	8′-11	9′-8
14′-0	0′-11	2′-0	2′-1	2′-4	2′-10	3′-6	4′-7	7′-0	8′-5	9′-2	10′-0
14′-6	0′-11	2′-1	2′-2	2′-5	2′-11	3′-8	4′-9	7′-3	8′-8	9′-6	10′-4
15′-0	0′-11	2′-2	2′-3	2′-6	3′-0	3′-9	4′-11	7′-6	9′-0	9′-10	10′-9
15′-6	1′-0	2′-3	2′-4	2′-7	3′-1	3′-11	5′-1	7′-9	9′-4	10′-2	11′-1
16′-0	1′-0	2′-4	2′-5	2′-8	3′-2	4′-0	5′-3	8′-0	9′-7	10′-6	11′-5
16′-6	1′-0	2′-4	2′-6	2′-9	3′-4	4′-2	5′-5	8′-3	9′-11	10′-10	11′-10
17′-0	1′-1	2′-5	2′-7	2′-10	3′-5	4′-3	5′-7	8′-6	10′-2	11′-2	12′-2
17′-6	1′-1	2′-6	2′-8	2′-11	3′-6	4′-5	5′-9	8′-9	10′-6	11′-6	12′-6
18′-0	1′-2	2′-7	2′-8	3′-0	3′-7	4′-6	5′-11	9′-0	10′-10	11′-10	12′-10
18′-6	1′-2	2′-8	2′-9	3′-1	3′-9	4′-8	6′-1	9′-3	11′-1	12′-2	13′-3
19′-0	1′-2	2′-9	2′-10	3′-2	3′-10	4′-9	6′-3	9′-6	11′-5	12′-6	13′-7
19′-6	1′-3	2′-9	2′-11	3′-3	3′-11	4′-11	6′-5	9′-9	11′-8	12′-10	13′-11
20′-0	1′-3	2′-11	3′-0	3′-4	4′-0	5′-0	6′-7	10′-0	12′-0	13′-2	14′-3
21′-0	1′-4	3′-0	3′-2	3′-6	4′-2	5′-3	6′-11	10′-6	12′-7	13′-10	15′-0
22′-0	1′-5	3′-2	3′-4	3′-8	4′-5	5′-6	7′-3	11′-0	13′-2	14′-5	15′-9
23′-0	1′-5	3′-3	3′-5	3′-10	4′-7	5′-9	7′-7	11′-6	13′-10	15′-1	16′-5
24′-0	1′-6	3′-5	3′-7	4′-0	4′-10	6′-0	7′-11	12′-0	14′-5	15′-9	17′-2
25′-0	1′-7	3′-7	3′-9	4′-2	5′-0	6′-3	8′-3	12′-6	15′-0	16′-5	17′-10
26′-0	1′-8	3′-9	3′-11	4′-4	5′-2	6′-6	8′-7	13′-0	15′-7	17′-1	18′-7
27′-0	1′-8	3′-10	4′-1	4′-6	5′-5	6′-9	8′-11	13′-6	16′-2	17′-9	19′-4
28′-0	1′-9	4′-0	4′-2	4′-8	5′-7	7′-0	9′-3	14′-0	16′-10	18′-5	20′-0
29′-0	1′-10	4′-2	4′-4	4′-10	5′-10	7′-3	9′-7	14′-6	17′-5	19′-1	20′-9
30′-0	1′-11	4′-4	4′-6	5′-0	6′-0	7′-6	9′-11	15′-0	18′-0	19′-9	21′-5

STIRRUPS IN CONCRETE BEAMS

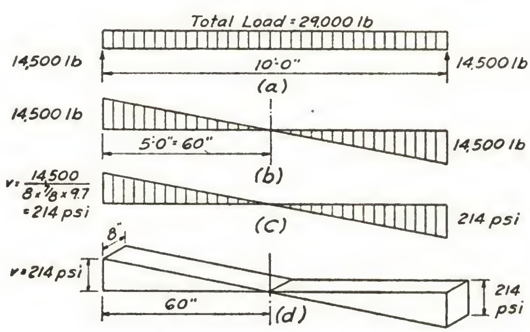
This section shows some simple methods for determining the size of stirrups in reinforced concrete beams and also their spacing, either read directly from a slide rule or, if preferred, from tables which are here presented.

Stirrups must be so designed that their total cross-sectional area provides, at f_v = 20,000 psi, sufficient strength to resist the force represented by the excess shear prism *abcde* ((e) in figure, page 87) and carried one beam depth beyond the point where web reinforcement is no longer required (ACI 801d). (Since the vertical component of diagonal tension in any distance *s* along the beam is proportional to the horizontal shear in that distance, *vbs*, the volume of the wedge or prism is proportional to, or represents, the vertical component of diagonal tension in one-half of the beam, and the portion of the prism above the v_c = 90 psi limit represents the load which must be carried by stirrups.) The stirrup diameter must be small enough to permit complete development in bond within the half-depth of the beam. The size and spacing must be such that when each stirrup is located at the centroid (approximately) of equal partial volumes of the excess shear prism, the spacing does not exceed $d/2$ (ACI 806),* and in addition stirrups must be carried a distance *d* beyond the theoretical distance *a*(ACI 801d).

The problem resolves itself mainly into simple procedures for locating stirrups at the centers of these partial volumes, as illustrated in the following examples:—

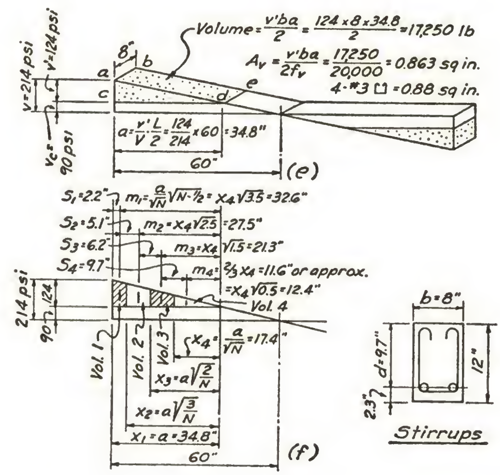
Example 1—Stationary Uniform Load; Triangular Shear Diagram. Determine the size and spacing of stirrups in an 8 x 12 in. (d = 9.7 in.) simple rectangular beam, uniformly loaded, neglecting the effect of bent-up tension bars.

Solution—The figure shows the load diagram (a) which produces an external shear diagram (b), a shear intensity diagram (c), with a longitudinal shear prism as shown in (d). In (e) on page 87 the longitudinal shear prism is divided into two sections, the un-



* "ACI" in this section refers to "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

STIRRUPS IN CONCRETE BEAMS



stippled portion showing the amount of shear taken by 3000 psi concrete at the 90 psi allowed by ACI 305a, leaving the stippled volume to be carried by web reinforcement at a stress of $f_v = 20{,}000$ psi. The computations on the diagram show how the required stirrup area of $A_v = 0.863$ sq in. was obtained, and is conveniently supplied by four U-shaped stirrups of #3 bar (0.88 sq in.). Although the bent-up longitudinal steel might care for a certain portion of this volume, it is usually best (and economical) to disregard the bent-up bars * because the designer can not foresee just what zone will be covered by the sloping portion of the bars, and, at best, the truss bars would replace only one or two stirrups. For the case of heavy girders, see Example 5.

In the figure above, the excess shear prism (representing that portion of the vertical component of diagonal tension which must be carried by the stirrups) is shown divided into four equal parts, each one representing the load on one stirrup. Theoretically the stirrups should be placed at the centroids of these volumes. It is entirely within the range of desired precision to consider that the upper set of dimensions (m values) locates these points. The subtractions indicated in the figure give the required spacings. The operation is carried through easily on a slide rule, using scratch paper for subtraction if required. The procedure is as follows:—

SET THE CROSS-HAIR AT THE VALUE OF $a = 34.8$ *in.* (*figure* (*e*) *above*) *ON THE D SCALE.*

SET THE VALUE OF N ON THE B SCALE AT THE CROSS-HAIR. The index is now at the value of x_4 on the D scale.

SET THE CROSS-HAIR AT THE VALUE OF $(N - \frac{1}{2})$ *ON THE B SCALE.* The cross-hair then indicates the value of m_1 on the D scale. The difference between a and m_1 is s_1, the distance of the first stirrup from the edge of the support.

SET THE CROSS-HAIR AT THE VALUE OF $(N - 1\frac{1}{2})$ *ON THE B SCALE.* The cross-hair then indicates the value of m_2 on the D scale; $m_1 - m_2$ gives the distance between the first two stirrups, s_2.

CONTINUE THUS TO A SETTING FOR THE CROSS-HAIR ON THE B SCALE OF 1.5. This gives m_3 in the figure, and $m_2 - m_3$ equals s_3.

FINALLY, MULTIPLY THE VALUE OF x_4 *by* ⅔ *TO GET* m_4 (or, if the strict sequence above is followed, by 0.5 on the B scale; $\sqrt{0.5} = 0.70$). Subtracting $m_3 - m_4$ equals s_4.

Thus s_1, s_2, s_3 and s_4 are 2.2, 5.1, 6.2 and 9.7 in., respectively, or closely enough 2, 5, 6, 9.

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* See pages 82-83 for bending of bars.

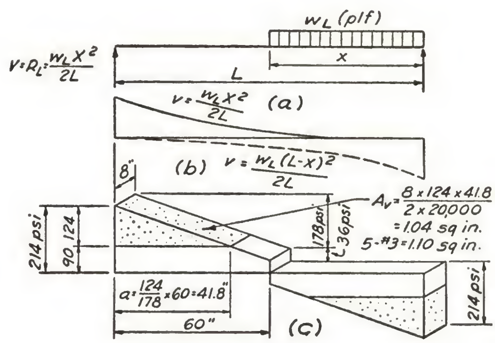
STIRRUPS IN CONCRETE BEAMS

Since ACI 806 requires a maximum spacing of approximately $d/2$ (about 4.9 in. in this case), it becomes necessary to add stirrups from the point where the maximum spacing is first encountered for the rest of the distance, resulting in 2, 4½, 4½, 4½, 4½, 4½, requiring a total of six stirrups, plus at least two more stirrups to satisfy ACI 801d. It might be better to use eight (or more) #2 stirrups spaced:—

"B"	8	7.5	6.5	5.5	4.5	3.5	2.5	1.5	0.5
"D"	34.8	33.7	31.4	28.8	26.1	23.0	19.4	15.1	8.7
Spacing		1.1	2.3	2.6	2.7	3.1	3.6	(4.3)	(6.4)
								4 4½ 4½	+ 4½ 4½

Alternative Solution of Spacing—Similar results can be obtained from the table on page 48. Having determined $a = 34.8$ and $N = 4$, take from the table $0.07a$, $0.16a$, $0.16a$ and $0.26a$ and multiply to obtain 2.4, 5.5, 5.5 and 9, or, say, 2, 5, 6, 9, which is the same as the 2, 5, 6, 9 obtained previously, then add two stirrups as described above to satisfy ACI 801d.

Example 2—Movable Live Loads. If the load and span remain the same as in Example 1 but two-thirds of the total load is live load capable of being removed, design the stirrups.

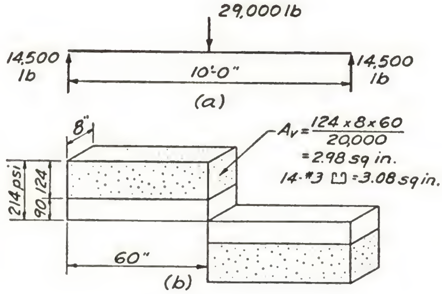


Solution—The figure above shows that the maximum live shear at any point becomes $V = w_L x^2/2L$; at midspan $V = w_L L/8$, or exactly one-quarter of the live end shear. The external shear diagram is shown in (c) with practically 36 psi at midspan. The inclined surface is taken as a plane, though actually it is slightly concave, the assumption being on the safe side. As before, compute the value of $a = 41.8$ in. and $A_v = 1.04$ sq in. This area might be made with 5-#3 stirrups if the spacings work out within the maximum allowable (and then include two more spaced at d/2 to satisfy ACI 801d):—

"B"	5	4.5	3.5	2.5	1.5	0.5
"D"	41.8	39.6	35.0	29.5	22.9	13.2
Spacing		2.2	4.6	(5.5)	(6.6)	(9.7)
				4	4 4	4 + 4 4

Example 3—Center-Concentrated Load; Rectangular Shear Prism. Given a center-concentrated load as shown in the figure on page 89, neglecting the dead weight of the beam, compute the excess shear volume, number of stirrups and their spacing.

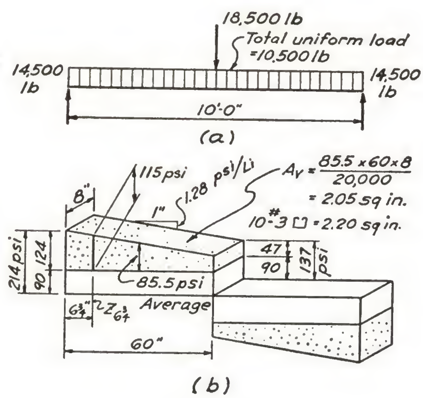
STIRRUPS IN CONCRETE BEAMS



Solution—A_v is computed on the figure as 2.98 sq in., requiring 14-#3 stirrups. The easiest method of spacing is to divide the 60-in. distance by fourteen stirrups, obtaining a constant spacing of 4¼ in., with a half-space at each end. (ACI 801d would not add any stirrups in this case.)

For future use, another method is illustrated, dividing the capacity of a stirrup by the longitudinal shear per running inch of beam. Each stirrup accounts for $0.22 \times 20{,}000 = 4400$ lb of longitudinal shear. The longitudinal shear amounts to $8 \times 124 = 992$ pli. The spacing becomes $4400/992 = 4.44$ in.

Example 4—Uniform-plus-concentrated Load; Trapezoidal Shear Prism. Design stirrups for the combination of concentrated and uniform loads shown in the figure below on an 8 x 12 in. rectangular beam.



Solution—The shear prism is shown and the area and make-up of web reinforcement are computed on the diagram. The problem is to space the ten stirrups at the centers of equal shear prisms. It is possible to work out a formula for doing this, but a cut-and-try solution is quicker and easier to apply. Each stirrup would account for $2 \times 0.11 \times 20{,}000 = 4400$ lb of longitudinal shear, but since 2.20 sq in. were provided

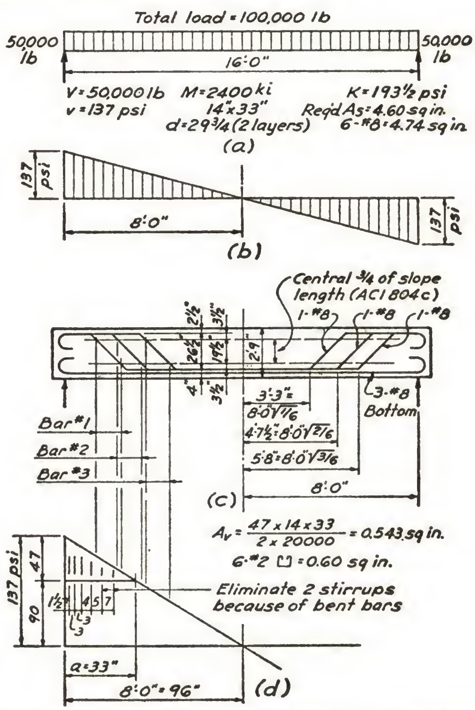
STIRRUPS IN CONCRETE BEAMS

for 2.05 sq in. required, use $\frac{2.05}{2.20} \times 4400 = 4100$ lb. In an 8 in. wide beam, this is equivalent to 513 lb per inch of thickness of beam stem. At the extreme end, the excess longitudinal shear is 124 psi, so that the first stirrup would be about one-half of $\frac{513}{124}$, or $2\frac{1}{4}$ in. from the support, where the longitudinal shear is $(124 - 2\frac{1}{4} \times 1.28)$, or 121 psi, requiring stirrups $\frac{513}{121} = 4\frac{1}{4}$ in. c/c. The second stirrup might be $6\frac{3}{4}$ in. from the support, where the shear intensity is $(124 - 6\frac{3}{4} \times 1.28)$, or 115 psi, requiring $\frac{513}{115} = 4\frac{1}{2}$ in. c/c. The third stirrup, at $11\frac{1}{4}$ in. from the support, with a shear of 109, requires a spacing of $\frac{513}{109} = 4\frac{3}{4}$ in. c/c. The probable location, excess longitudinal shear and spacing for all of the stirrups are tabulated below:—

Stirrup Number	1	2	3	4	5	6	7	8	9	10
Dist from Support (in.)	2¼	6¾	11¼	16	21½	27	33	39½	47	55½
Longitudinal Shear (psi)	121	115	109	104	96	89	81½	73½	63½	53
Spacing (in.)	2¼	4½	4¾	5	5¼	5¾	6¼	7	8	9¾

Since the maximum spacing can not exceed $d/2 = 4\frac{3}{4}$ in. (ACI 806), additional stirrups will be required anyhow, but this serves to illustrate the cut-and-try method. A variation of the above is to compute the spacing at either end and the center of the trapezoid as $\frac{4100}{8 \times 124} = 4\frac{1}{8}$; $\frac{4100}{8 \times 47} = 11$; $\frac{4100}{8 \times 85\frac{1}{2}} = 6$; then space between by approximation as 2¼, 4½, 4¾, 5, 5¼, 5¾, 6¼, 7, 8, 9¾. (ACI 801d would not add any stirrups in this case.)

Example 5—Girders using Bent-up Bars and Stirrups for Web Reinforcement. Design web reinforcement for a heavy rectangular reinforced concrete girder 14 x 33 in. ($d = 29\frac{3}{4}$ in., two layers of steel) carrying 100,000 lb on a span of 16 ft, using truss bars where possible to eliminate stirrups.



STIRRUPS IN CONCRETE BEAMS

Solution—This procedure is practical only on heavy girders, where it is deemed advisable to detail the girder to scale rather than tabulate in a schedule. The figure on page 90 shows an elevation of the girder, the computation of moment, $R = \frac{M}{bd^2}$, and tension reinforcement; while (b) shows the shear intensity diagram. In (c), an elevation of the girder is drawn and, from a parabolic moment curve, the points determined at which individual truss bars can be spared and bent up. The zone within which the diagonal bars are to be considered effective, that is, the middle three-quarters of the bent portion (ACI 804a), is indicated. In (d), the shear intensity diagram is divided into a lower portion (90 psi on the concrete) and an upper triangle to be carried by web reinforcement. On this triangle, stirrups are designed and spaced as described in Ex. 1 above. Then, by scale or computation, those stirrups which fall within the zone of effective width of inclined bars are automatically eliminated. A check should be made to see that the effective area of the truss bar is at least equal to that of the eliminated stirrups. The requirement of ACI 801d is pretty well fulfilled by the sloping portion of the bar that bends up nearest to midspan.

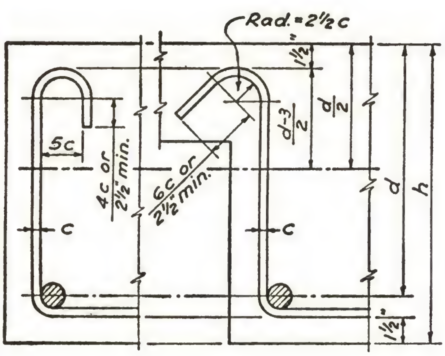
See page 48 for table of stirrup spacings.

MINIMUM TOTAL BEAM DEPTH (h) IN INCHES FOR VERTICAL STIRRUPS TO DEVELOP FULL STRESS

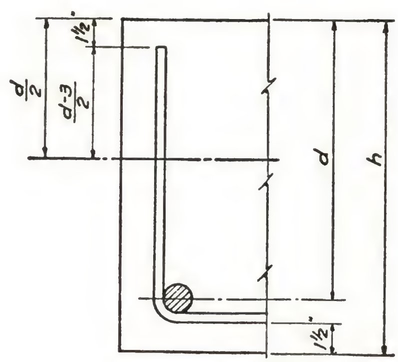
f'_c = 3000 psi
f_v = 20,000 psi

u = 300 psi (deformed)
120 psi (plain)

Stirrups With Hooks



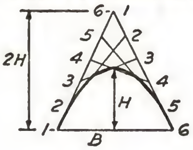
Stirrups Without Hooks

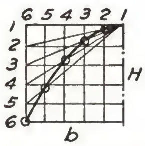


f'_c	Plain #2	Deformed #3	Deformed #4	Deformed #5	f'_c	Plain #2	Deformed #3	Deformed #4	Deformed #5
2000	19	15	18	22	2000	33	24	31	37
2500	17	13	16	19	2500	28	21	26	31
3000	15	12	14	16	3000	24	18	23	27
3750	13	11	13	15	3750	21	16	20	24

PROPERTIES OF PARABOLA

TO DRAW A SYMMETRICAL PARABOLA





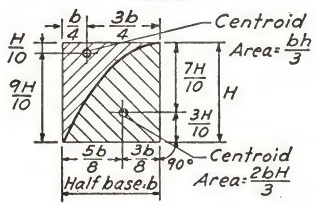
Method I

1. Given Base B, Height H.
2. Construct isosocles triangle, Base B, Height 2H.
3. Divide each equal leg into same number of equal parts.
4. Connect as shown.
5. These connecting lines are tangent to inscribed parabola.

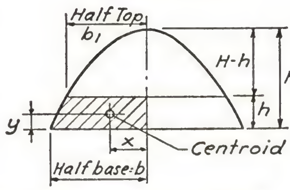
Method II

1. Given half-base b, Height H.
2. Divide inclosing rectangle with same number of vertical and radiating lines.
3. Intersections of similarly numbered verticals and rays are points on the curve.

AREAS AND CENTROIDS OF PARABOLIC FIGURES



PARABOLIC SEGMENT



$$h = \frac{H}{b^2}(b^2 - b_1^2)$$

$$b_1 = b\sqrt{\frac{H-h}{H}}$$

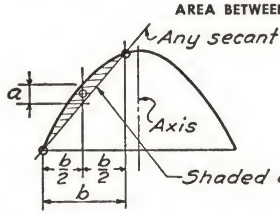
$$H = \frac{hb^2}{b^2 - b_1^2}$$

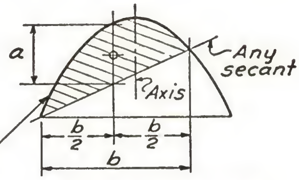
$$\text{Area} = \tfrac{2}{3}h\left(\frac{b^3 - b_1^3}{b^2 - b_1^2}\right)$$

$$x = \tfrac{3}{8}\left(\frac{b^2h - b_1^2h_1}{bh - b_1h_1}\right)$$

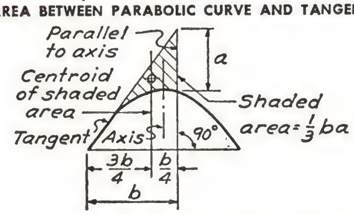
$$y = \tfrac{3}{10}\left[\frac{bh^2 - b_1h_1^2}{bh - b_1h_1}\right]$$

AREA BETWEEN PARABOLIC CURVE AND SECANT

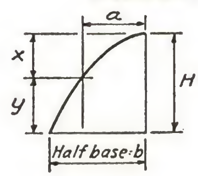




AREA BETWEEN PARABOLIC CURVE AND TANGENT



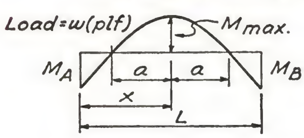
INTERMEDIATE ORDINATES



$$x = H\sqrt{\frac{a}{b}}$$

$$y = H - x$$

POSITIVE MOMENT CONSISTENT WITH GIVEN NEGATIVE VALUES AT ENDS



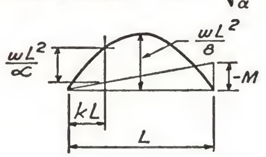
$$X = \frac{L}{2} + \frac{M_B - M_A}{wL}$$

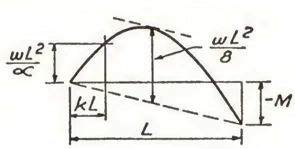
$$+M_{max} = \frac{wL^2}{8} + \frac{M_B - M_A}{2} + \frac{(M_A - M_B)^2}{2wL^2}$$

$$a = \sqrt{\frac{L^2}{4} + \frac{M_B + M_A}{w} + \frac{(M_A - M_B)^2}{w^2L^2}}$$

ABSCISSA FOR GIVEN ORDINATE; RELATION POSITIVE AND NEGATIVE MOMENTS

For Given Positive Moment $\quad k = \sqrt{\frac{2}{\alpha}}$ ——— ——— ——— $-M = (\tfrac{1}{2} - k)wL^2$





For Given Negative Moment $\quad k = \tfrac{1}{2} - \frac{M}{wL^2}$ ——— ——— $\alpha = \frac{2}{k^2}$ ——— ——— ——— $+M = \frac{wL^2}{\alpha}$

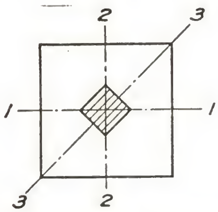
KERN OF REINFORCED CONCRETE COMPRESSION MEMBERS

If the load on a column is slightly off the centroidal axis, the compressive stress on one side is reduced and on the other side increased. With increasing eccentricity, the reduced compression drops to zero and then changes to gradually increasing tension. The kern (or core) of a reinforced concrete compression member delimits the area of the cross section within which a force parallel to the longitudinal axis produces compression all across the column section; a force outside of the kern produces tension on part of the section.

HOMOGENEOUS SECTION

Square (for P applied on axis 1-1 or 2-2):—

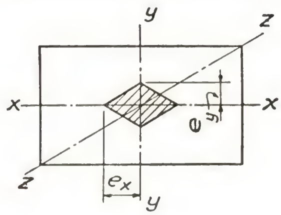
$$f = \frac{P}{A} \pm \frac{Pec}{I}; \quad \text{when } f = 0, \quad \frac{P}{A} = \frac{Pec}{I} \quad \text{and} \quad e = \frac{I}{cA} = \frac{bd^3}{12\left(\frac{d}{2}\right)bd} = \frac{d}{6},$$



hence the "middle-third" rule, that the applied load should fall within the middle third of the axis length to avoid tension.

It can be shown that the shaded area bounded by lines connecting these four points mark the kern within which the point of application of the load must lie if tension is to be avoided.

Rectangle (for P applied on axis x or y):—



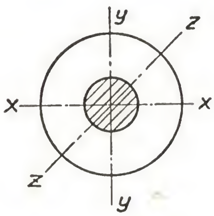
$$e_x = \frac{db^3}{12\left(\frac{b}{2}\right)bd} = \frac{b}{6}$$

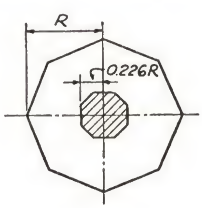
$$e_y = \frac{bd^3}{12\left(\frac{d}{2}\right)bd} = \frac{d}{6}$$

It can be shown that the shaded area bounded by lines connecting these four points mark the kern within which the point of application of the load must lie if tension is to be avoided.

Circle:— $e = \dfrac{I}{cA} = \dfrac{\pi r^4}{4 \cdot r\pi r^2} = \dfrac{r}{4}$, so the kern is a circle of radius $\dfrac{r}{4}$.

Octagon:—If the octagon is inscribed within a circle of radius R, the kern is $0.226R$ across the flats.

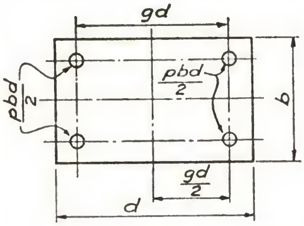




KERN OF REINFORCED CONCRETE COMPRESSION MEMBERS

REINFORCED CONCRETE SECTIONS

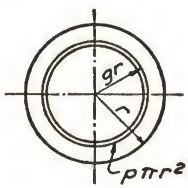
Rectangular Reinforced Tied Concrete Column (equal reinforcement in each of the faces perpendicular to the plane of bending):—



$$e = \frac{I}{cA} = \frac{\dfrac{bd^3}{12} + npbd\left(\dfrac{gd}{2}\right)^2}{\left(\dfrac{d}{2}\right)(bd + npbd)} = \frac{d}{6}\left[\frac{1 + 3npg^2}{1 + np}\right]$$

The kern of a rectangular reinforced column with equal bars in two faces will lie outside that of an unreinforced column if the value in the bracket exceeds unity, i.e., if $(1 + 3npg^2) > (1 + np)$, or $3g^2 > 1$, i.e., $g > \sqrt{1/3}$, or >0.58.

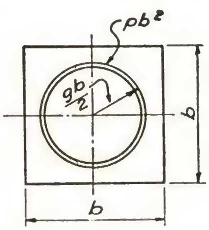
Round Reinforced Concrete Column (reinforcement considered as a closed ring):—



$$e = \frac{I}{cA} = \frac{\dfrac{\pi r^4}{4} + np\pi r^2\left(\dfrac{g^2r^2}{2}\right)}{(r)(\pi r^2 + np\pi r^2)} = \frac{r}{4}\left[\frac{1 + 2npg^2}{1 + np}\right]$$

The kern of a reinforced round column will lie outside that of an unreinforced column if the value in the bracket exceeds unity, i.e., if $(1 + 2npg^2) > (1 + np)$, or $2g^2 > 1$, i.e., $g > \sqrt{1/2}$, or >0.707.

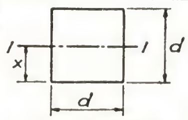
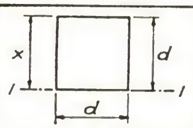
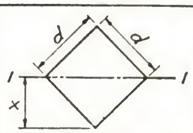
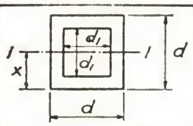
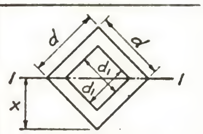
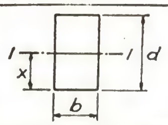
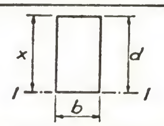
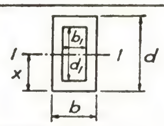
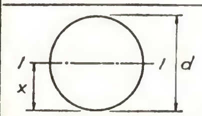
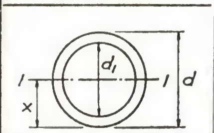
Square Spirally Reinforced Concrete Column:—



$$e = \frac{I}{cA} = \frac{\dfrac{b^4}{12} + npb^2\left(\dfrac{g^2b^2}{8}\right)}{\dfrac{b}{2}(b^2 + npb^2)} = \frac{b}{6}\left[\frac{1 + \dfrac{3npg^2}{2}}{1 + 2np}\right]$$

The kern of a spirally reinforced square column will always lie inside that of an unreinforced concrete column since the value in the bracket is always less than unity.

PROPERTIES OF SECTIONS

Section	Area	Axis to Extreme Fiber	Moment of Inertia	Radius of Gyration	Section Modulus
	A	x	I	$r = \sqrt{I \div A}$	$S = I \div x$
	sq. in.	in.	in.^{4}	in.	in.^{3}
	d^2	$\frac{d}{2}$	$\frac{d^4}{12}$	$\frac{d}{\sqrt{12}} = 0.2887d$	$\frac{d^3}{6}$
	d^2	d	$\frac{d^4}{3}$	$\frac{d}{\sqrt{3}} = 0.5773d$	$\frac{d^3}{3}$
	d^2	$\frac{d}{\sqrt{2}} = 0.7071d$	$\frac{d^4}{12}$	$\frac{d}{\sqrt{12}} = 0.2887d$	$\frac{d^3}{6\sqrt{2}} = 0.1179d^3$
	$d^2 - d_1^2$	$\frac{d}{2}$	$\frac{d^4 - d_1^4}{12}$	$\sqrt{\frac{d^2 + d_1^2}{12}}$ $\sqrt{\frac{1}{12}} = 0.2887$	$\frac{d^4 - d_1^4}{6d}$
	$d^2 - d_1^2$	$\frac{d}{\sqrt{2}} = 0.7071d$	$\frac{d^4 - d_1^4}{12}$	$\sqrt{\frac{d^2 + d_1^2}{12}}$ $\sqrt{\frac{1}{12}} = 0.2887$	$\frac{d^4 - d_1^4}{6d\sqrt{2}}$ $\frac{1}{6\sqrt{2}} = 0.1179$
	bd	$\frac{d}{2}$	$\frac{bd^3}{12}$	$\frac{d}{\sqrt{12}} = 0.2887d$	$\frac{bd^2}{6}$
	bd	d	$\frac{bd^3}{3}$	$\frac{d}{\sqrt{3}} = 0.5773d$	$\frac{bd^2}{3}$
	$bd - b_1d_1$	$\frac{d}{2}$	$\frac{bd^3 - b_1d_1^3}{12}$	$\sqrt{\frac{bd^3 - b_1d_1^3}{12(bd - b_1d_1)}}$	$\frac{bd^3 - b_1d_1^3}{6d}$
	$\frac{\pi d^2}{4} = 0.7854d^2$	$\frac{d}{2}$	$\frac{\pi d^4}{64} = 0.0491d^4$	$\frac{d}{4}$	$\frac{\pi d^3}{32} = 0.0982d^3$
	$\frac{\pi(d^2 - d_1^2)}{4}$ $\frac{\pi}{4} = 0.7854$	$\frac{d}{2}$	$\frac{\pi(d^4 - d_1^4)}{64}$ $\frac{\pi}{64} = 0.0491$	$\frac{\sqrt{d^2 + d_1^2}}{4}$	$\frac{\pi(d^4 - d_1^4)}{32d}$ $\frac{\pi}{32} = 0.0982$

DEAD WEIGHTS OF FLOORS, CEILINGS AND ROOFS, IN POUNDS PER SQUARE FOOT

	Weight (psf)
FLOORINGS:—	
Cement finish, per inch of thickness	12
Cinder concrete fill, per inch of thickness	8
3″ creosoted wood blocks on ½″ mortar base	21
2″ creosoted wood blocks on ½″ mortar base	17
3″ creosoted wood blocks on ⅛″ mastic bed	12
2″ creosoted wood blocks on ⅛″ mastic bed	9
⅞″ hardwood floor on sleepers clipped to concrete without fill	5
1½″ terrazzo floor finish directly on slab	19
1½″ terrazzo floor finish on 1″ mortar bed	30
1″ terrazzo finish on 2″ concrete bed	38
¾″ ceramic or quarry tile on ½″ mortar bed	16
¾″ ceramic or quarry tile on 1″ mortar bed	22
¼″ linoleum or asphalt tile directly on concrete	1
¼″ linoleum or asphalt tile on 1″ mortar bed	12
¾″ mastic floor	9
Hardwood flooring, ⅞″ thick	4
Subflooring (soft wood), ¾″ thick	2½
Gypsum slab, per inch of thickness	6
Asphalt mastic finish, 1½ in. thick	18
CEILINGS:—	
¾″ plaster directly on concrete, blocks or tile	5
¾″ plaster on metal lath furring	8
¾″ gypsum plaster on metal lath and channel suspended ceiling construction	10
Plaster on rock lath and channel ceiling construction	6
Acoustical fiber tile directly on concrete blocks or tile	1
Acoustical fiber tile on rock lath and channel ceiling construction	5
Acoustical fiber tile on suspended wood furring strips	3
ROOFS:—	
Five-ply felt and gravel (or slag)	6½
Three-ply felt and gravel (or slag)	5½
Five-ply felt composition roof, no gravel	4
Three-ply felt composition roof, no gravel	3
Asphalt strip shingles	3
Cement tile	16
Slate, ¼″ thick	9½
Slate, ½″ thick	19
Sheathing, ¾″ thick, Yellow Pine	3½
Sheathing, ¾″ thick, Spruce or Hemlock	2½
Skylight with galvanized iron frame, ¼″ wire glass	7
Gypsum, per inch of thickness	4
Poured gypsum on steel rails, per inch of thickness	5
Light-weight fill or insulation, porous glass, vermiculite, etc., per inch of thickness	1 to 2
Light-weight fill or insulation, cinder concrete, per inch of thickness	8
Spanish tile (laid)	9 to 12
Shingle-type clay tile	12 to 14
Metal deck (20 gauge)	2¼
Metal deck (18 gauge)	3
Corrugated metal (20 gauge)	1½
Flat cement tile, per inch of thickness	13

(For weights of concrete joists, see pages 142 to 154.)

Portland cement may be shipped in bulk or in sacks weighing 94 lb per sack, ordinarily considered 1 cu ft (4 sacks = 376 lb are referred to as a barrel).

DEAD WEIGHTS OF WALLS AND PARTITIONS, IN POUNDS PER SQUARE FOOT

	Weight (psf)		
	Un-plastered	One Side Plastered	Both Sides Plastered
WALLS:—			
4″ brick wall	40	45	50
9″ brick wall	80	85	90
13″ brick wall	120	125	130
17″ brick wall	160	165	170
21″ brick wall	205	210	215
25″ brick wall	245	250	255
4″ concrete block	28	33	38
6″ concrete block	36	41	46
8″ concrete block	51	56	61
12″ concrete block	59	64	69
4″ hollow light-weight block (tile or cinder)	19	24	29
6″ hollow light-weight block (tile or cinder)	22	27	32
8″ hollow light-weight block (tile or cinder)	33	38	43
12″ hollow light-weight block (tile or cinder)	44	49	54
4″ brick, 4″ hollow concrete block backing	68	73	—
4″ brick, 8″ hollow concrete block backing	91	96	—
4″ brick, 12″ hollow concrete block backing	119	124	—
4″ terra cotta tile	25	30	35
8″ terra cotta tile	33	38	43
12″ terra cotta tile	45	50	55
4″ glass block	20	—	—
Windows, glass, frame and sash	8	—	—
Porcelain enamel on sheet steel	3	—	—
Structural glass, per inch of thickness	15	—	—
4″ stone	55	—	—
Asbestos hardboard (corrugated), per ¼″ of thickness	3	—	—
4″ brickwork with 4″ hollow tile backing	60	65	—
4″ brickwork with 8″ hollow tile backing	75	80	—
PARTITIONS:—			
3″ clay tile	17	22	27
4″ clay tile	18	23	28
6″ clay tile	25	30	35
8″ clay tile	31	36	41
10″ clay tile	35	40	45
3″ gypsum block	10	15	20
4″ gypsum block	13	18	23
5″ gypsum block	16	21	26
6″ gypsum block	17	22	27
2″ solid plaster	—	—	20
2 x 4 studs, or metal studs, lath and ¾″ plaster	—	—	18
Steel partitions	4	—	—
½″ plaster on gypsum block or clay tile	—	4	8

Partitions are sometimes arbitrarily allowed for as 12, 15, or 18 psf of floor area, which, at best, is not very precise. It is better to take off and weigh the partitions in a panel or two for any building of importance.

DEAD WEIGHTS OF MASONRY, IN POUNDS PER CUBIC FOOT

MASONRY:—	*Weight (pcf)*
Cinder concrete fill	60
Concrete, cinder	100
Concrete, slag	130
Concrete, stone	144
Concrete, reinforced stone	150
Brick masonry, soft	110
Brick masonry, common	125
Brick masonry, pressed	140
Dry rubble masonry, sandstone, bluestone	110
Dry rubble masonry, limestone, marble	125
Dry rubble masonry, granite, gneiss	130
Mortar rubble masonry, sandstone, bluestone	130
Mortar rubble masonry, limestone, marble	150
Mortar rubble masonry, granite, gneiss	155
Ashlar sandstone, bluestone	140
Ashlar limestone, marble	160
Ashlar granite, gneiss	165

BEARING CAPACITIES OF SOILS
(Boston Building Code)

Class	Material	Allowable Bearing Value (Tons Per Sq Ft)
1	Massive bedrock without laminations, such as granite, diorite and other granite rocks; and also gneiss, trap rock, folsite and thoroughly cemented conglomerates, such as the Roxbury Puddingstone, all in sound condition (sound condition allows some cracks).	100 *
2	Laminated rocks, such as slate and schist, in sound condition (some cracks allowed).	35
3	Shale in sound condition (some cracks allowed).	10
4	Residual deposits of shattered or broken bedrock of any kind except shale.	10
5	Hardpan.	10
6	Gravel, sand-gravel mixtures, compact.	5
7	Gravel, sand-gravel mixtures, loose; sand, coarse, compact.	4
8	Sand, coarse, loose; sand, fine, compact.	3
9	Sand, fine, loose.	1
10	Hard clay.	6
11	Medium clay.	4
12	Soft clay.	1
13	Rock flour or any deposit of unusual character not provided for herein.	(Values to be fixed by the Commissioner)

* But not greater than the value of the footing resting upon it.

BEARING VALUES OF MASONRY WALLS

	Allowable Bearing (psi)
3000 psi concrete *	750
2000 psi concrete *	500
1500 psi concrete *	375
Hard-burned common brick in cement mortar	200
Soft common brick in cement mortar	150
Hard-burned common brick in lime mortar	150
Soft common brick in lime mortar	120
Load-bearing back-up tile (cells vertical) †	150
Concrete blocks ‡ in cement mortar	150
Cinder blocks ‡ in cement mortar	120
Stone masonry in cement mortar	250

* Values given are for full area loaded and may be increased to a maximum 50% greater if load covers one-third or less of the area; other values interpolated.

† Non-load-bearing tile or tile with cells horizontal are not desirable materials for supporting loads.

‡ Meeting ASTM C90-52; increase 75% if solid units.

STRENGTH OF MATERIALS
Building Materials

Material	Average Ultimate Stress (psi)			Safe Working Stress (psi)			Modulus of Elasticity (psi)
	Compression	Tension	Bending	Compression	Bearing	Shearing	
Masonry, granite	......	......	......	420	600		
Masonry, limestone, bluestone	......	......	......	350	500		
Masonry, sandstone	......	......	......	280	400		
Masonry, rubble	......	......	......	140	250		
Masonry, brick, common	10000	200	600				
Ropes, cast steel hoisting	......	80000					
Ropes, standing, derrick	......	70000					
Ropes, manila	......	8000					
Stone, bluestone	12000	1200	2500	1200	1200	200	7,000,000
Stone, granite, gneiss	12000	1200	1600	1200	1200	200	7,000,000
Stone, limestone, marble	8000	800	1500	800	800	150	7,000,000
Stone, sandstone	5000	150	1200	500	500	150	3,000,000
Stone, slate	10000	3000	5000	1000	1000	175	14,000,000

BUILDING CODE REQUIREMENTS FOR LIVE LOADS IN POUNDS PER SQUARE FOOT

Occupancy	Codes								
	Basic Building Code BOCA 1955	Am. Std. Bldg. Code 1955 Nat. Bureau Stds.	Nat. Bd. of Fire Underwriters 1955	Internat'l Conference Bldg. Officials 1955	New York 1957	Chicago 1956	Philadelphia	Detroit	Southern Building Code Congress Southern Std. Bldg. Code 1954
Dwellings, apartment and tenement houses, hotels, club houses, hospitals and places of detention:									
Dwellings, private rooms and apartments	40 [30]	40	40 [30]	40	40 [11]	40	40	40	40 [43]
Public corridors, lobbies and dining rooms	100 [29]	100	100 [29]	100	100	100	100	100	100
School buildings:									
Class rooms and rooms for similar use	60 [27]	40	40	40 [7]	60 [12]	40	50 [25]	50 [25]	40
Corridors and public parts of the building	100	100	100	100	100	100	100	100	100
Theaters, assembly halls etc.									
Auditoriums with fixed seats	60	60	60	50	75 [13]	60	60 [26]	60	50
Lobbies, passageways, gymnasiums, grandstands, stages and auditoriums or places of assemblage without fixed seats	100 [8]	100	100 [8]	100 [8]	100	100	100	100	100 [8]
Stage floor	150	150	150	125		150		150	150
Office building:									
Office space	50 [2,3]	80	80	50 [2,3]	50 [11]	50 [21,3]	60	50 [34]	50
Corridors and other public places	100	100	80	100	100 [42]	100	100	100	100
Workshops, factories and mercantile establishments:									
Manufacturing—light	125	125	125 [2]	75	120	100 [41]	120 [28]	125 [35]	100
Manufacturing—heavy				125	120 [41]	100 [41]	200 [28]	125	150
Storage—light	125		125 [2]	125	120	100 [41]	120-150 [28]	125 [36]	125
Storage—heavy	250		250 [2]	250	120 [41]	100 [41]	200 [28]	250	250
Stores—retail	75 [20]	100 [52]	75 [2]	75	75 [15]	100 [41]	100 [28]	100 [35]	75
Stores—wholesale	125	125	125 [2]	100	75 [15]	100 [41]	100 [28]	125	100
Garages:									
All types of vehicles	175 [16]		100-200 [45]	100 [9]	175 [14]	100 [9]	100 [4]	175 [37]	120 [9]
Passenger cars only	75 [16]	100	100 [2]	50 [9]	75 [17]	50 [23]	75	80 [38]	75
Libraries									
Stack rooms in schools	150	150	150	125				150	
Reading rooms		60	60					60	
All stairs and fire escapes, except in private residences	100 [47]	100	100 [48]	100	100	100	100	100 [39]	100
Roofs (flat)	20 [49]		20	20 [5]	40	25	30	30	20
Sidewalks	250 [4]	250	250 [4]	250	300 [18]		150 [31]	250	200 [4]
Wind	Min 20 [10]		15-40 [50]	12-50 [1]	0-20 [19]	20 [24]	15-25 [32]	20 [40]	10-40 [51]

BUILDING CODE REQUIREMENTS FOR LIVE LOADS IN POUNDS PER SQUARE FOOT

Notes:

[1] 15 psf up to 60 ft high, 20 psf over 60 ft.
[2] Or 2000 lb on any space 2½ feet square.
[3] Where partitions are subject to change add 20 psf to all other loads.
[4] Or 8000 lb concentrated.
[5] If area is 200 to 600 sf use 16 psf, over 600 sf, 12 psf.
[7] 60 for library reading rooms and 125 for stackrooms.
[8] 150 for armories.
[9] Or concentrated rear wheel of loaded truck in any position.
[10] Increase 0.025 psf for each foot above 100 ft.
[11] Including corridors.
[12] For rooms with fixed seats or, by special permission, other small rooms. 120 for library stackrooms.
[13] 60 for churches.
[14] 6000 lb concentrated. Trucking space 100% max. wheel load, 12,000 lb conc. min., 175 psf on floor construction, 120 psf on beams and girders.
[15] 100 for entire first floor.
[16] Or 2000 lb concentrated. Trucking space, 150% max. wheel load; 175 psf on floor construction, 120 psf on beams and girders.
[17] Or 2000 lb concentrated.
[18] Or 12,000 lb concentrated for driveways over sidewalks.
[19] 20 psf from top down to 100 ft level, zero below; 30 psf on tanks, stacks and exposed structures.
[20] 100 psf on floor at grade, upper floors 75 psf.
[21] Or 2000 lb concentrated on any space 2½ feet square.
[22] Or 3000 lb concentrated on any space 4 feet square.
[23] 100 on first floor and alternate of 2000 lb on area 2½ feet square.
[24] 20 for buildings less than 300 ft high; add 0.025 psf per ft above 300 ft.
[25] Only school class rooms with fixed seats. (Removable seats 80 psf)
[26] Churches only.
[27] Fixed seats, 60 psf; removable seats, 100 psf.
[28] Every floor beam 4000 lb concentrated.
[29] Corridors in hotels, hospitals and multifamily dwellings (except corridors serving public rooms in hotels), 60 psf; corridors in one and two family dwellings 40 psf.
[30] On first floor, 40 psf; upper floors, 30 psf.
[31] Interior courts, sidewalks, etc., not accessible to a driveway.
[32] 15 psf up to 50 ft high, 20 psf from 50 to 200 ft, 25 psf over 200 ft high. Roofs over 30°, 20 psf on windward side, 10 psf on leeward.
[34] Above first floor including corridors.
[35] 125 for first floor.
[36] 150 for first floor.
[37] Or 2500 lb concentrated on area 6 inches square with such concentrations spaced alternately 2 ft 4 in. and 4 ft 8 in. in one direction and 5 ft and 10 ft in the other direction.
[38] Only structures with clear head room of 8 ft 6 in. or less. Or 1500 lb concentrated spaced as in 37.
[39] 50 for dwellings and apartments under 3 stories.
[40] For buildings less than 500 ft high.
[41] The minimum for storage or manufacturing is 120 psf (100 psf, Chicago), but floors must be designed for any heavier loads contemplated and for any concentrations.
[42] Including entire first floor but not including corridors on floors used for offices.
[43] 30 for one-story, one and two family dwellings.
[44] 10 for portions below 40 ft and 20 for portions above 40 ft.
[45] 150 psf for passenger cars; trucks-150 psf (3 to 10 tons incl. load), 200 psf (over 10 tons incl. load); concentrated load-150 per cent max. wheel load for passenger cars, 125 per cent max. axle load for trucks.
[46] 75 psf for open parking decks.
[47] 40 psf for one and two family dwellings; 300 lb concentrated load at center of any stair tread.
[48] 300 lb on 2½ ft square at any location.
[49] Not less than 30 psf when subject to snow loads.
[50] 15 psf for building height $<$ 30 ft; increase to 40 psf for building height $\geqq$ 1200 ft.
[51] 10 psf for portions below 30 ft; 40 psf for portions above 400 ft in Southern Inland Regions. Varies from 25 to 50 psf for Southern Coastal Regions.
[52] First floor 100, upper floors 75.

RECOMMENDED LIVE LOADS FOR STORAGE WAREHOUSES

U. S. Department of Commerce, National Bureau of Standards

Material	Weight per Cubic Ft of Space (lb)	Ht of Pile (ft)	Weight per Sq Ft of Floor (lb)	Recmd. Live Load (psf)
Building Materials				
Asbestos	50	6	300	
Bricks, Building	45	6	270	
Bricks, Fire Clay	75	6	450	
Cement, Natural	59	6	354	
Cement, Portland	72-105	6	432-630	300
Gypsum	50	6	300	to
Lime and plaster	53	5	265	400
Tiles	50	6	300	
Woods, Bulk	45	6	270	
Drugs, Paints, Oils, etc.				
Alum, pearl, in barrels	33	6	198	
Bleaching powder in hogsheads	31	3½	102	
Blue vitriol, in barrels	45	5	226	
Glycerine, in cases	52	6	312	
Linseed oil, in barrels	36	6	216	
Linseed oil, in iron drums	45	4	180	
Logwood extract, in boxes	70	5	350	
Rosin, in barrels	48	6	288	200
Shellac, Gum	38	6	228	to
Soaps	50	6	300	300
Soda ash, in hogsheads	62	2¾	167	
Soda, Caustic, in iron drums	88	3⅜	294	
Soda, Silicate, in barrels	53	6	318	
Sulphuric acid	60	1⅝	100	
Toilet articles	35	6	210	
Varnishes	55	6	330	
White lead paste, in cans	174	3½	610	
White lead, dry	86	4¾	408	
Red lead and Litharge, dry	132	3¾	495	
Dry Goods, Cotton, Wool, etc.				
Burlap, in bales	43	6	258	
Carpets and rugs	30	6	180	
Coir Yarn, in bales	33	8	264	
Cotton, in bales, American	30	8	240	
Cotton, in bales, Foreign	40	8	320	
Cotton bleached goods in cases	28	8	224	
Cotton Flannel, in cases	12	8	96	
Cotton Sheeting, in cases	23	8	184	
Cotton Yarn, in cases	25	8	200	
Excelsior, compressed	19	8	152	200
Hemp, Italian, compressed	22	8	176	to 250
Hemp, Manila, compressed	30	8	240	
Jute, compressed	41	8	328	
Linen Damask, in cases	50	5	250	
Linen Goods, in cases	30	8	240	
Linen Towels, in cases	40	6	240	
Silk and Silk Goods	45	8	360	
Sisal, compressed	21	8	168	
Tow, compressed	29	8	232	
Wool, in bales, compressed	48			
Wool, in bales, not compressed	13	8	104	
Wool, Worsteds, in cases	27	8	216	

RECOMMENDED LIVE LOADS FOR STORAGE WAREHOUSES

U. S. Department of Commerce, National Bureau of Standards

Material	Weight per Cubic Ft of Space (lb)	Ht of Pile (ft)	Weight per Sq Ft of Floor (lb)	Recmd. Live Load (psf)
Groceries, Wines, Liquors, etc.				
Beans, in bags	40	8	320	
Beverages	40	8	320	
Canned Goods, in cases	58	6	348	
Cereals	45	8	360	
Cocoa	35	8	280	
Coffee, Roasted, in bags	33	8	264	
Coffee, Green, in bags	39	8	312	
Dates, in cases	55	6	330	
Figs, in cases	74	5	370	
Flour, in barrels	40	5	200	250
Fruits, Fresh	35	8	280	to
Meat and Meat Products	45	6	270	300
Milk, Condensed	50	6	300	
Molasses, in barrels	48	5	240	
Rice, in bags	58	6	348	
Sal Soda, in barrels	46	5	230	
Salt, in bags	70	5	350	
Soap Powder, in cases	38	8	304	
Starch, in barrels	25	6	150	
Sugar, in barrels	43	5	215	
Sugar, in cases	51	6	306	
Tea, in chests	25	8	200	
Wines and Liquors, in barrels	38	6	228	

Material	Weight per Cubic Ft of Space (lb)	Ht of Pile (ft)	Weight per Sq Ft of Floor (lb)	Recmd. Live Load (psf)
Hardware, Etc.				
Automobile Parts	40	8	320	
Chain	100	6	600	
Cutlery	45	8	360	
Door Checks	45	6	270	
Electrical Goods and Machinery	40	8	320	
Hinges	64	6	384	
Locks, in cases, packed	31	6	186	
Machinery, Light	20	8	160	
Plumbing, Fixtures	30	8	240	300
Plumbing, Supplies	55	6	330	to
Sash Fasteners	48	6	288	400
Screws	101	6	606	
Shafting steel	125			
Sheet Tin, in boxes	278	2	556	
Tools, Small, Metal	75	6	450	
Wire Cables, on reels			425	
Wire, Insulated Copper in coils	63	5	315	
Wire, Galvanized iron, in coils	74	4½	333	
Wire, Magnet, on spools	75	6	450	
Miscellaneous				
Automobile tires	30	6	180	
Automobiles, uncrated	8		64	
Books (solidly packed)	65	6	390	
Furniture	20			
Glass and Chinaware, in crates	40	8	320	
Hides and Leather, in bales	20	8	160	
Hides, Buffalo, in bundles	37	8	296	
Leather and Leather goods	40	8	320	
Paper, Newspaper, and Strawboards	35	6	210	
Paper, Writing and Calendered	60	6	360	
Rope, in coils	32	6	192	
Rubber, Crude	50	8	400	
Tobacco, bales	35	8	280	

FIRE RESISTIVE RATINGS OF CONCRETE CONSTRUCTION

The fire rating is the time, in hours, which a given member or construction will withstand the standard fire in accordance with the "Standard Methods of Fire Tests of Building Construction and Materials," ASTM E119. The following conditions determine the end point of the test and the rating is established by the first end point reached.

Bearing walls and floors must support their design load throughout the test and must not allow the passage of flames or hot gasses sufficient to ignite cotton waste held on the surface. An end point is reached when the specimen fails to support its load or when the temperature on the surface away from the fire rises an average of 250°F or 325°F at any one point. In practically all tests of concrete walls or floors the controlling end point has been this temperature rise on the unexposed surface.

The end point for columns, beams and girders is the time at which the member will no longer carry its design load.

FLOORS

Type	Description	Rating
Reinforced Concrete Slab	4½" thick—¾" protection reinforcement—expanded slag aggregate	4 hr.
	6" thick—1" protection reinforcement—air cooled slag aggregate	4 hr.
	6½" thick—1" protection reinforcement—other aggregates	4 hr.
	5½" thick—1" protection reinforcement—other aggregates	3 hr.
	6" thick with electrical raceways and junction boxes—¾" protection reinforcement	3 hr.
	4½" thick—¾" protection reinforcement	2 hr.
	3" thick—¾" protection reinforcement	1 hr.
Reinforced Concrete Joist	3" top slab *—¾" protection reinforcement—ceiling 1" vermiculite-gypsum or perlite-gypsum plaster on metal lath	4 hr.
	2" top slab *—¾" protection reinforcement—ceiling ¾" vermiculite-gypsum or perlite-gypsum plaster on metal lath	3 hr.
	3" top slab—¾" protection reinforcement	1 hr.
	*—Increase slab thickness 2" where electrical raceways and junction boxes occur.	
Reinforced Concrete Joists with Concrete Block or Tile Fillers	4" lightweight aggregate concrete block with 2" concrete topping—¾" protection reinforcement	3 hr.
	6" clay tile with 2" concrete topping—⅝" sand-gypsum plaster	3 hr.
	4" clay tile with 1½" concrete topping—⅝" sand-gypsum plaster	2 hr.

CONCRETE WALLS

Type	Description	Rating
Reinforced Concrete Walls	6½" thick—1" protection reinforcement	4 hr.
	5½" " —1" protection reinforcement	3 hr.
	4½" " —¾" protection reinforcement	2 hr.
	3" " —¾" protection reinforcement	1 hr.
	Note: portland cement stucco or plaster or gypsum plaster may be substituted for an equivalent thickness of concrete.	

FIRE RESISTIVE RATINGS OF CONCRETE CONSTRUCTION

HOLLOW CONCRETE MASONRY WALLS

Type	Description	Minimum Equivalent Thickness Inches,* for Ratings of			
		4 hr.	3 hr.	2 hr.	1 hr.
Hollow Concrete Masonry Units	Coarse aggregate, expanded slag, or pumice..	4.7	4.0	3.2	2.1
	Coarse aggregate, expanded clay or shale...	5.7	4.8	3.8	2.6
	Coarse aggregate, limestone, cinders or unexpanded slag........................	5.9	5.0	4.0	2.7
	Coarse aggregate, calcareous gravel........	6.2	5.3	4.2	2.8
	Coarse aggregate, siliceous gravel...........	6.7	5.7	4.5	3.0

* Equivalent thickness is the average thickness of the solid material in the wall. It may be found by taking the total volume of a wall unit, subtracting the volume of core spaces, dividing this by the area of the face of the unit. Where walls are plastered or faced with brick the thickness of plaster or brick may be included in determining the equivalent thickness.

Where combustible members are framed into the wall, the wall must be of such thickness or be so constructed that the thickness of solid material between the end of each member and the opposite face of the wall, or between members set in from opposite sides, will be not less than 93% of the thickness shown in the table.

COLUMNS, BEAMS, GIRDERS AND TRUSSES

Type	Protection of Reinforcement	Rating
Reinforced Concrete Columns	1½″ concrete—coarse aggregate limestone, calcareous gravel, trap rock or blast furnace slag—12″ or larger column	4 hr.
	2″ concrete—coarse aggregate granite, sandstone, or siliceous gravel—16″ or larger column	4 hr.
	1½″ concrete—coarse aggregate granite, sandstone or siliceous gravel—16″ or larger column	3 hr.
Reinforced Concrete Beams, Girders and Trusses	1½″ concrete	4 hr.
	1″ concrete	1 hr.

VOLUME OF CONCRETE IN COLUMNS

Round Columns and Caps

Column Diam (in.)	Column Area (sq in.)	Vol Shaft Per Vert Ft (cu ft)	Volume of Cap outside of Shaft (cu ft), Diameter of Cap = D: 6′-0	5′-6	5′-0	4′-6	4′-0	3′-6
12	113	.79	29.62	22.75	17.02	12.31	8.54	5.61
14	154	1.07	28.88	22.09	16.43	11.79	8.09	5.23
16	201	1.40	28.07	21.35	15.77	11.22	7.60	4.82
18	255	1.77	27.17	20.55	15.06	10.60	7.08	4.38
20	314	2.18	26.20	19.69	14.30	9.95	6.52	3.94
22	380	2.64	25.17	18.77	13.50	9.26	5.95	3.48
24	452	3.14	24.09	17.81	12.66	8.55	5.37	3.02
26	531	3.69	22.95	16.81	11.80	7.82	4.78	2.56
28	616	4.28	21.77	15.78	10.92	7.09	4.19	2.12
30	707	4.91	20.56	14.73	10.02	6.35	3.61	1.70
32	804	5.59	19.32	13.66	9.12	5.62	3.04	1.31
34	908	6.31	18.06	12.58	8.22	4.90	2.51	.95
36	1018	7.07	16.79	11.50	7.33	4.20	2.00	....
38	1134	7.88	15.51	10.42	6.46	3.53	1.53	....
40	1257	8.73	14.23	9.36	5.61	2.89	1.10	....

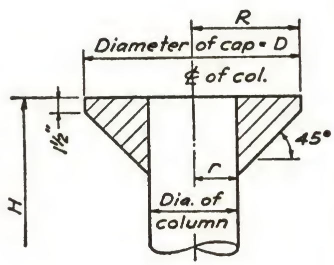
To obtain the volume of column and cap:—

1. Multiply Dimension "H" in feet by "Volume of Shaft" (Col. 3).
2. From "Dia. of Column" and "Dia. of Cap" find volume of cap.
3. The sum of Items 1 and 2 equal the volume of one column.

Formula for Volume of Cap outside of Shaft:—

$$V = \frac{\pi}{1728}\left[1\tfrac{1}{2}(R^2 - r^2) + \tfrac{1}{3}(R^3 - r^3) - r^2(R - r)\right]$$

V = Volume of Cap in cu ft. R = Radius of cap in in.
r = Radius of Shaft in in.



Square Columns

Column Size (in.)	Volume in Cubic Feet Per Vert Ft	Column Size (in.)	Volume in Cubic Feet Per Vert Ft
10 x 10	.70	25 x 25	4.34
11 x 11	.84	26 x 26	4.69
12 x 12	1.00	27 x 27	5.06
13 x 13	1.17	28 x 28	5.44
14 x 14	1.36	29 x 29	5.84
15 x 15	1.56	30 x 30	6.25
16 x 16	1.78	31 x 31	6.67
17 x 17	2.01	32 x 32	7.11
18 x 18	2.25	33 x 33	7.56
19 x 19	2.51	34 x 34	8.03
20 x 20	2.78	35 x 35	8.51
21 x 21	3.06	36 x 36	9.00
22 x 22	3.36	37 x 37	9.51
23 x 23	3.67	38 x 38	10.03
24 x 24	4.00	39 x 39	10.56
		40 x 40	11.11

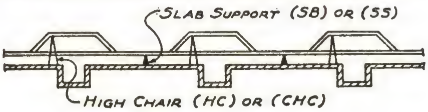
SPECIFICATIONS FOR PLACING BAR SUPPORTS

All reinforcing steel shall be accurately located in the forms, and firmly held in place, before and during the placing of concrete, by means of wire supports adequate to prevent displacement during the course of construction and to keep the steel at a proper distance from the forms.

Bar supports are to be sufficient in number and sufficiently heavy to carry properly the steel they support. The wire sizes and number of supports shall not be less than the following:

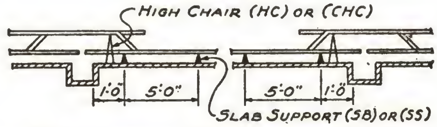
ONE WAY SLAB CONSTRUCTION

Bars continuous over more than one panel.



If clear distance between beams is more than 6′-0″ use 2 supports.

Bars not continuous.



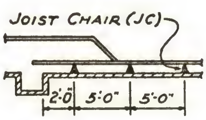
Slab Supports—End Spacing 1′-0″ Max. Maximum Intermediate Spacing 5′-0″.

Individual High Chairs—(HC)—spaced not more than 4′-0″ centers with not less than #5 support bars. Continuous High Chairs—(CHC)—may be substituted for High Chairs and #5 support bar.

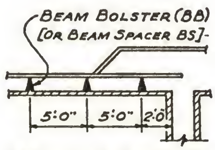
JOIST-BEAM-GIRDER CONSTRUCTION

Beam and Joist Construction

Beam bars #9 and smaller.



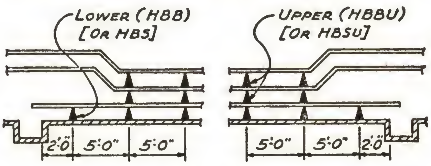
JOIST BAR SUPPORTS



BEAM BAR SUPPORTS

Heavy Beam and Girder Construction

Beam or Girder with bars larger than #9.



BEAM BAR SUPPORTS

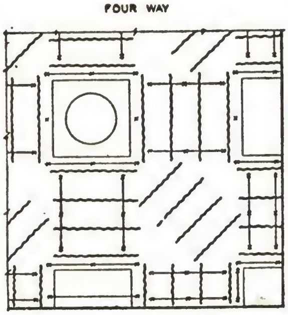
Maximum end spacing 2′-0″—Maximum intermediate spacing 5′-0″ for both lower and upper layers.

No supports are to be provided for temperature mesh or bars in concrete joist floors. It is recommended that temperature bars be tied and spaced with #2 bars 4′-2″ centers at right angles to temperature bars.

(Some designers support ends of bent bars in joists with #3 bar at each side of and parallel to supporting beam or wall, held above form by individual chairs—(BC) spaced approximately 25″ on centers.)

FLAT SLAB CONSTRUCTION

FOUR WAY

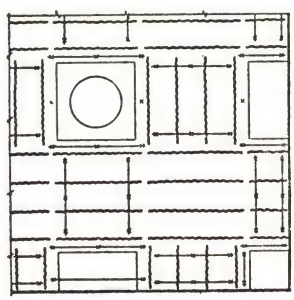


~~~Designates Slab Supports (SB) or (SS).
—Designates #5 Support Bar.
X—Designates High Chairs—(HC).

Slab Supports—For spans over 24′-0″ use 5 slab supports where 4 are shown on diagram.

Continuous High Chairs—CHC—May be substituted for High Chairs and #5 support bars.

TWO WAY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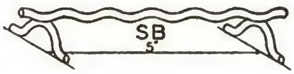
BAR SUPPORT SPECIFICATIONS AND STANDARD NOMENCLATURE

Symbol	Bar Support	Top Wire *	Legs *	Description
SB	Slab Bolster	No. 4 Corrugated	¾″ high—No. 7 Over ¾″—No. 5	Legs spaced 5″ centers—Corrugations vertical or flat spaced 1″ centers—Heights up to 2″. **Stocked in ¾″, 1″, 1½″, 2″ heights and 5 and 10 foot lengths.**
SBR	Slab Bolster with Runners	No. 4 Corrugated	Same as SB	Same as SB with No. 7 Wire Runners
SS	Slab Spacer	No. 5 Smooth	Same as SB	Legs spaced to provide supporting leg under each bar. Minimum leg spacing 4″—Heights up to 2″. **Fabricated to order.**
BB	Beam Bolster	No. 7 Smooth	No. 7	All legs spaced 2″ centers—Maximum height 3″. **Stocked in 1″, 1½″, 2″ heights, in 5 foot lengths.**
HBB	Heavy Beam Bolster	No. 4 Smooth	No. 4	Same as BB except maximum height 5″.
BBU	Beam Bolster Upper	No. 7 Smooth	No. 7	All legs spaced 2″ centers—Maximum height 3″. **Stocked in 1″, 1½″, 2″ heights, in 5 foot lengths.**
HBBU	Heavy Beam Bolster Upper	No. 4 Smooth	No. 4	Same as BBU except maximum height 5″. **Fabricated to order.**
† BS	Beam Spacer	No. 7 Smooth	No. 7	Fabricated to order for desired bar spacing and beam width—Maximum height 3″.
† HBS	Heavy Beam Spacer	No. 4 Smooth	No. 4	Same as BS except maximum height 5″.
† BSU	Beam Spacer Upper	No. 7 Smooth	No. 7	Fabricated to order for desired bar spacing and beam width—Maximum height 3″.
† HBSU	Heavy Beam Spacer Upper	No. 4 Smooth	No. 4	Same as BSU except maximum height 5″.
JC	Joist Chair	No. 8	No. 8	**Made and stocked only in 4, 5, 6 inch widths and ¾″, 1″, 1½″ heights.**
BC	Bar Chair	No. 8	No. 8	**Made and stocked only in ¾″, 1″, 1½″ and 1¾″ heights.**
HC	Individual High Chairs	No. 5 for 2″ to 4″ No. 4 over 4″ to 6″ No. 2 over 6″ to 9″ No. 0 over 9″		**Stocked in ¼″ increments from 2″ to 6½″.**
CHC	Continuous High Chairs	No. 2 for 2″ to 6″ No. 0 for over 6″	Same as HC	All legs 12″ centers. **Fabricated to order.**
CHCU	Continuous High Chairs Upper	Same as CHC	Same as CHC	Same as CHC with No. 5 wire runners.
HCHC	Heavy Continuous High Chairs	Same as CHC	Same as CHC	Legs 8″ centers. **Fabricated to order.**

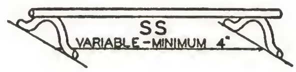
* AS&W wire gauges indicated in this table are the minimum sizes to be used.
† Not included in standard specifications and nomenclature.

BAR SUPPORTS SPECIFICATIONS AND STANDARD NOMENCLATURE

Wire Specifications—Standard Bright Basic Wire

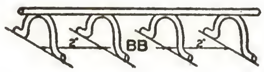


SLAB BOLSTER



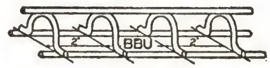
SLAB SPACER

BAR CHAIR



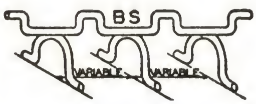
BEAM BOLSTER

SLAB BOLSTER WITH RUNNERS

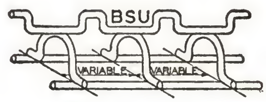


BEAM BOLSTER UPPER

HEAVY BEAM BOLSTER (HBB & HBBU) SIMILAR

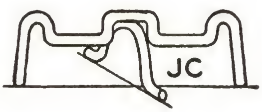


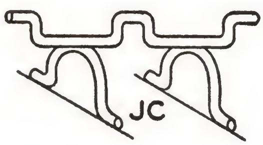
BEAM SPACER



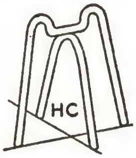
BEAM SPACER UPPER

HEAVY BEAM SPACERS (HBS & HBSU) SIMILAR

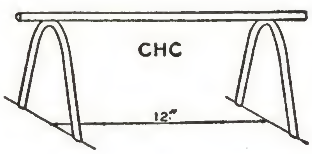




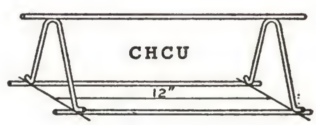
JOIST CHAIR—Three Standard Types Shown



HIGH CHAIR



CONTINUOUS HIGH CHAIR



CONTINUOUS HIGH CHAIR UPPER

HEAVY CONTINUOUS HIGH CHAIR (HCHC). Legs 8"o.c.

Legs can be furnished of galvanized wire or hot dipped galvanized after forming, when required, for small additional charge.

Types BC, HC, CHC, HCHC are supplied with straight legs as shown but can be furnished with upturned legs if so ordered.

Types SB, SS, BB, and BS are supplied with upturned legs as shown but can be had on special order with straight end bearing legs. This type of leg is designated by the suffix "A," i.e., slab bolster with end bearing leg is "SBA."

GENERAL SUGGESTIONS FOR DRAWINGS OF REINFORCED CONCRETE STRUCTURES

The following list of suggestions outlines a number of important items which should be covered on design drawings either by details or notes. Numerical values and sizes given are not intended to represent a standard of practice but merely to illustrate that the designer should specify clearly his individual preference or need.

1. All detailing, fabrication and erection of reinforcing bars, unless otherwise noted, must follow the ACI "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-latest)."
2. All concrete not otherwise specified to be stone, gravel or slag concrete to test 3000 psi in standard 6 x 12 in. cylinders at 28 days.
3. Not less than 5½ sacks of cement shall be used per cubic yard of concrete regardless of the strength obtained, not over 6½ gallons of water per sack of cement, and not over 4 in. slump.
4. Reinforcing bars (*except for column verticals when called for as hard grade*) are to be intermediate grade, deformed, new billet steel meeting ASTM A15 (latest) *; or rail steel meeting ASTM A16 (latest).*
5. All bars (excepting #2 size) are to have deformations meeting ASTM A305 (latest).*
6. All bars are to be supported in the forms and spaced with wire bar supports meeting the requirements of the ACI "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315-latest)." †
7. All solid slabs ‡ (unless otherwise specified) shall have temperature steel consisting of (*designer should state his requirements*).
8. All slabs of concrete joist construction ‡ (unless otherwise specified) shall have (*designer should here state the size and spacing of temperature bars or the gauge and mesh of welded wire fabric that he wants*).
9. All concrete slabs on the ground ‡ that are not otherwise provided for shall have temperature reinforcement consisting of (*designer supply size and spacing of bars or gauge and mesh of welded wire fabric*).
10. Welded wire fabric must have end laps of one full mesh (tip to tip of longitudinal wires) and edge laps obtained by overlapping longitudinal selvage wires and wiring all laps securely together. Welded wire fabric should extend into supporting beams and walls for anchorage unless an expansion joint is called for.
11. Reinforce all walls (unless otherwise specified) with #4 bars @ 12 in. c/c horizontal and vertical. (*Designer should cover wall reinforcement with descriptions on the drawings and use this note only for miscellaneous walls not otherwise provided for.*)
12. Lap all splices 24 bar diameters (12 in. minimum) unless otherwise called for (24 diameters for top bars with 12″ of concrete underneath); bend all horizontal wall bars and all wall footing bars 1′-0″ around all corners.
13. Provide at least two #4 bars in top of wall footing under door and other openings, 4′-0″ longer than the opening.

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* See pages 398-410.

† See pages 107-109.

‡ Designer should clearly specify any other places to be reinforced with welded wire fabric or temperature bars.

GENERAL SUGGESTIONS FOR DRAWINGS OF REINFORCED CONCRETE STRUCTURES

14. Provide dowels in wall footings equivalent in size and number to vertical steel extending 24 bar dia. into footing and 24 bar dia. into wall. (*Designer should check that this can be done.*)
15. All areaways (unless otherwise specified) are to be doweled to main building with #4 bars @ 1′-0″ c/c, hooked or embedded 24 bar diameters into wall.
16. Lintels over all openings in interior partitions not otherwise covered are to be of precast concrete with thickness equal to the wall thickness. Depth shall be 8 in. for spans up to 6′-0″, reinforced with 1-#4 bar for each 4 in. of wall thickness.
17. Removable steel forms for concrete joist construction shall be at least 16 gauge, smooth, not corrugated, 20 or 30 in. wide, with narrow widths where required and with ends tapered where called for at a standard taper of 2 in. each side in 3′-0″.
18. Unless otherwise shown, form around all openings, vents, etc. with double joists and headers if complete information on location is provided to supplier by the time shop drawings of form layouts are approved.
19. Structural concrete over forms of concrete joist construction is to be a minimum of 2 in. thick, but not less than what is called for in the schedules.
20. Designer should indicate all special framings in concrete joist construction, including double or extra heavy joists under partitions,* distribution ribs, special spacings under batteries of toilets, and so on. Quite a few suggestions appear in the safe load tables on the succeeding pages.
21. Joists up to 8 in. wide parallel and adjacent to walls or beams need no reinforcement, providing they are integral with a bearing of concrete on the beam or wall. If independent of the wall or beam, they should have one full set of joist bars. When integral parallel joists are more than 8 in. in width, use one full set of joist bars.
22. Submit shop drawings for reinforcing steel and shop drawings for forms in concrete joist construction to the engineer in triplicate and obtain approval before fabricating.
23. Designer should indicate, either on standard diagrams under the schedules or in notes, the bending of bars, amount of cover above top bars and under bottom bars, amount of lap of bottom bars at interior support, clear distance between outside of concrete and stirrup leg, amount of embedment at walls, cover over column bars, amount of lap at splices, and any other pertinent information.†
24. In masonry bearing walls, no chases, risers, conduits or toothing of masonry shall occur within 1′-6″ of center-line of beam bearing or concentration.
25. Brick pilasters in backup walls where required for bearing are to be bonded into adjoining masonry.

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* Note that a heavier joist each side of a partition, possibly with thickened concrete over a line of shallower pans, interferes less with vertical stands of piping in partitions than does an extra wide joist directly underneath the partition.

† Many designers allow a stated tonnage of bars to be used as directed in the field for special conditions.

GENERAL SUGGESTIONS FOR DRAWINGS OF REINFORCED CONCRETE STRUCTURES

26. All interior exposed corners of concrete columns and beams to have a ¾ in. x 45° chamfer.
27. When foundation walls span from basement to first floor, both basement floor slab and first floor slab shall be in place before any backfill is placed.
28. In two-way solid slabs, place short-span bars in the bottom layer.
29. Reinforced concrete beams are to have 8 in. bearing on walls, unless otherwise noted.
30. Solid slabs are to have at least 4 in. bearing on masonry walls, unless otherwise noted.
31. Where concrete beams frame into structural steel and no other detail is shown, structural steel fabricator shall provide at least two ¾ in. round anchor bolts with double nuts through the web or flanges of supporting steelwork, in addition to seat angles.
32. Provide two #3 stirrup tie bars in top of all concrete beams that have stirrups and do not have other top steel available for holding stirrups.
33. All reinforcing bars shall be securely wired together in the forms. Two-way mats of steel shall be tied at alternate intersections both ways; column ties and beam stirrups shall be tied sufficiently to hold them securely in place.

10 STEPS TO A SUCCESSFUL DESIGN

1. Study the framework of the building as a whole.

2. Prepare alternative freehand sketches, comparing various methods of framing.

3. Establish column centers to come in partitions, to clear door and window openings, and to provide economical framing. Spacings of from 14 to 15 feet up to 20 to 25 feet are recommended.

4. Take rough preliminary sizes either from this handbook or by that rule of thumb which suggests making beam depth one inch per foot of span and width one-half of that—increasing somewhat for unusually heavy loads, decreasing for unusually light loads.

5. Make quantity surveys and cost comparisons of all practicable schemes. This can be done quickly and roughly and still with a fair degree of accuracy.

6. Select that compromise which achieves the best balance between low cost of the building and minimum interference with desired facilities.

7. Preliminary framing sketches, made freehand on thin paper, can be printed and distributed to architects, mechanical engineers, and others to establish the general program and eliminate unnecessary changes on finished drawings.

8. Have a sense of comparative values. A stair header has relatively little effect on the overall cost, but a whole line of spandrel beams on many stories can become a large item.

9. In planning the building visualize how forms would be constructed. For economy, keep beams and columns simple, without haunches, brackets, widened ends or offsets.

10. Select a beam size suitable for the average load and span, using same breadth-width combination for as many load-span combinations as possible, varying reinforcement only; select steel form depth and use repeatedly from basement to roof regardless of load; use one or two column sizes per floor and use same size for three or four consecutive stories, and then reduce by several inches off one dimension of the column, next time off the other dimension.

(The following suggestions adapted from "A Manual of Standard Practice for Reinforced Concrete Construction," 1957, of the Concrete Reinforcing Steel Institute will repay careful reading.

STANDARD DETAILS OF DESIGN

A few brief suggestions regarding standard practice may assist the designer in giving his clients the most economical and satisfactory structure that can be designed.

It is recognized that absolute uniformity is neither possible nor desirable. It is felt, however, that while completed structures may differ widely from each other, the application of the principle of standardization in methods, and in the design of the individual parts of the various structures will result in economy of time and cost.

No attempt will be made to cover all the items in a building. However, a few basic principles are proposed.

Design. It is recommended that standard methods of design be followed, as given in the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," published by the American Concrete Institute, except where unusual conditions require departure from this practice.

Code of Standard Practice. It is recommended that the designer examine the Code of Standard Practice of the Concrete Reinforcing Steel Institute so as to be familiar with the practices and customs current in the industry. It will rarely be found necessary to vary from the principles incorporated in this Code.

Sizes of Bars and Spirals. It is recommended that the standard sizes of bars and spirals as specified in the Code of Standard Practice, in accordance with Simplified Practice Recommendations of the United States Department of Commerce, be observed. (See page 3.)

This gives the designer a sufficient range of sizes from which to select the steel areas. The number of different sizes used in any one structure should be held to a minimum.

Grade of Steel. When new billet reinforcing steel is specified, it is recommended (except for column verticals) that it be of intermediate grade in accordance with the current specifications of the American Society for Testing Materials, Designation A15. Rail Steel should meet ASTM Designation A16. (See pp. 398-410.)

Length of Bars. It is recommended that as few different lengths as possible be used. This expedites shipment of steel and greatly simplifies storing and handling in the field. Lengths should be given to the nearest inch only, and, where practicable, to the nearest 3 in. Where it is important that the length called for be exact, a note to this effect should appear opposite the item on the bar list.

Beams and Girders. As far as practicable, the spacing, widths and depths of beams and girders should be uniform throughout the structure. Consideration should be given the relative costs of steel, concrete and form work when determining their width and depth.

Flat Slabs. Where flat slab design is used, the dimensions of the column capital and the size of the dropped panel should be unchanged throughout the

building. It is recommended that the diameter of the column capital equal $.225l$ and the length of the dropped panel equal $.33l$.

Height of Stories. It is recommended that as many stories as possible in a building be of the same height. Where this is not possible, the lower stories should have the greater height.

Columns. Exterior columns should be the same width for the entire height of the structure. The space between columns should be of such size that an opening can be made to accommodate standard sizes of steel window sash.

The cross-sectional area of exterior columns should remain constant for at least two stories. Where the required area may be decreased, the change should be made in thickness only, the outside surface and the width of the column remaining constant, the inside surface only being set back.

Freestanding interior columns may be circular in cross-section. The diameter should be in even inches and changes should be made by intervals of two inches. Interior columns should be as small as is consistent with the structural requirements, and as far as practicable, the diameters of all columns should be constant throughout a single story.

The spacing of columns from center to center should be uniform. Column reinforcement should generally be of as large bars as is consistent with good practice.

Continuous spiral hooping should be used in preference to isolated hoops. Column steel should be spliced by allowing the column bars of the story below to project above the floor slab at least twenty (20) bar diameters for intermediate and hard grade new billet or rail steel deformed bars and forty (40) diameters for plain bars, unless adequate butt connections are provided to resist all stresses.

Footings. Where considerable variations exist in the depths to which different footings must be carried, a pedestal or pier on top of the footing may be used to reach the level at which the columns start. The connection of columns to the footing or pedestal should be made with dowels of the same size and number as the vertical bars in the column above. If footing depth is insufficient to develop the dowels, an additional quantity of smaller dowels should be used to transmit the entire stress in the steel by bond.

The number of different sizes of footings should be reduced to a minimum.

Forms. Building designs must be carried out so as to provide the maximum amount of repetition with the use of the minimum amount of forms. Owing to the constantly increasing cost, and to the decreasing quantity available, the conservation of form lumber requires attention.

It will often be found that steel forms will cost less than lumber when used throughout a season or when leased for each individual structure.

The greater uniformity of work built with steel forms will recommend their use to the average engineer.

Fabrication of Reinforcing Steel. It is recommended that all reinforcing steel be shop fabricated and so specified, as operations can be performed with greater accuracy by the special machinery in the shop.

SAFE CARRYING CAPACITY OF SOLID SLABS

Based upon "Building Code Requirements for Reinforced Concrete (ACI 318-56)," these tables give the safe carrying capacity (total load less dead weight of slab) in pounds per square foot for solid reinforced concrete slabs (a) with approximately balanced reinforcement ($p = 0.0136$) and (b) with approximately half that amount ($p = \frac{2}{3}\%$) for the one set of stresses, viz., $f_s = 20{,}000$ psi, $f_c = 1350$ psi, and for four conditions of end restraint:—(1) single spans simply supported, (2) end spans free on one end and continuous on the other, (3) interior spans continuous on both ends, and (4) interior spans of capacity approximately equal to that of end spans of equal thickness when $l'' = l'$ and $l'' = 1.2\,l'$. (See figure on page 125.)

Safe carrying capacities were obtained by computing the least safe total load as determined by shear or bond or flexure, and by deducting the weight of the slab itself to obtain the safe superimposed load. The tabulated capacity includes live load, finishes, ceilings and everything but the dead weight of the structural concrete.

Limitations as to stresses, restraints, number of spans continuous, amount of fireproofing, bending of bars, etc. are given in part on the general data sheets preceding each table or pair of tables, and in part at the head of each table.

Bond and diagonal tension values are based upon deformed bars meeting ASTM A305. Plain round bars and deformed bars not meeting A305 will not give sufficient bond resistance to develop these loads.

In continuous spans, the top steel in the tables is computed for the adjoining span being equal to 1.2 l' to obtain the maximum negative moment. The adjoining span may, by Code, be as small as 0.833 l', in which case the total top steel may be reduced to 70 per cent of the total given in the tables. For intermediate values of adjoining span, it is sufficiently accurate to prorate between these values.

In continuous spans with balanced reinforcement for positive moment, the concrete near midspan is stressed to its maximum value under the Code. Since the negative moment is considerably greater than the positive, the concrete in the bottom at the support would be overstressed unless the bottom bars were extended into the support as shown on the sketches preceding each table to provide reinforcement at the higher compressive stress allowed in the ACI Code (Art. 706b), viz., double the elastic value but not in excess of the allowable tension value. It is important that this detail be followed in scheduling designs from these tables.

ACI 318-56-701(c) establishes moment factors for cases where the unit live load does not exceed three times the unit dead load. If the ratio is greater, the effect of unbalanced panel loads may require more accurate analysis (see pages 66-80). The dead load in such analyses includes not only the weight of the concrete slab (which is here deducted from the total load to obtain the safe superimposed load), but any ceilings, floor finishes, partitions, and similar immovable features. Hence it is not practicable to indicate in these tables the points where the unit live load is exactly equal to three times the unit dead load.

SAFE CARRYING CAPACITY OF SOLID SLABS

Temperature reinforcement in structural floor and roof slabs is scheduled using deformed bars meeting ASTM A305. If plain bars are used, increase the quantity 25 per cent, which is best done by decreasing the spacing to 80 per cent of that given.

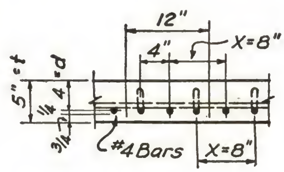
Bar supports are to be furnished as per pages 107 to 109.

Since there is frequently a progressive deflection in concrete slabs due to time yield, it is often advisable (although the ACI Code omits this consideration) to limit the span-depth ratio, especially for floor slabs. These tables indicate the point where $l'/t = 32$. The safe carrying capacities are computed for longer spans for use on roofs or where deflection is not a serious item.

For those who may wish to know how the tables were computed or desire to extend them beyond their present range, detailed computations are presented in the following typical examples:—

Example I—*Single Span, Approximately Balanced Reinforcement*—For the table on page 123, determine the safe carrying capacity on spans,* l', of 10 and 15 feet of a slab whose thickness (t) = 5 in.; reinforcement = #4 bars @ 4 in. c/c; weight of slab = 63 psf; steel percentage = 1.25%.

Unlike direct design, tables are worked in reverse, determining the total capacity of a definite slab for shear, bond and moment, selecting the smallest value and deducting the dead weight.



$$d = 5 - \tfrac{3}{4} - (\tfrac{1}{2} \times \tfrac{1}{2}) = 4 \text{ in.}$$

$$p = \frac{0.20}{4 \times 4} = 0.0125$$

From page 34, for $p = 0.0125$, take $k = 0.390$; $j = 0.870$; $R_c = 229.2$; $R_s = 217.5$. These can also be computed from the formulas on page 23.

Shear—$V = v_c bjd = 90 \times 12 \times 0.870 \times 4 = 3758 \text{ lb} = wl'/2$

When $l' = 10'\text{-}0''$, $w = \dfrac{2 \times 3758}{10} = 751$ psf. Subtracting 63 gives 688 psf.

$= 15'\text{-}0''$, $w = \dfrac{2 \times 3758}{15} = 501$ psf. Subtracting 63 gives 438 psf.

Bond—$V = u\Sigma ojd = 300 \times 1.571 \times \dfrac{12}{8} \times 0.870 \times 4 = 2460 \text{ lb} = wl'/2$

When $l' = 10'\text{-}0''$, $w = \dfrac{2 \times 2460}{10} = 492$ psf. Subtracting 63 gives 429 psf.

$= 15'\text{-}0''$, $w = \dfrac{2 \times 2460}{15} = 328$ psf. Subtracting 63 gives 265 psf.

Moment—$M = Rbd^2 = 217.5 \times 12 \times 4 \times 4 = 41{,}760 \text{ lb-in.} = wl'^2 12/8$

When $l' = 10'\text{-}0''$, $w = \dfrac{8 \times 41{,}760}{12 \times 10 \times 10} =$ 278 psf. Subtracting 63 gives 215 psf.

$= 15'\text{-}0''$, $w = \dfrac{8 \times 41{,}760}{12 \times 15 \times 15} =$ 123 psf. Subtracting 63 gives 60 psf.

As given in the table on page 123.

Temperature Steel—$A_t = 0.002bt = 0.002 \times 12 \times 5 = 0.120$ sq in. per ft width

#3 @ 11 in. = 0.120 sq in. per ft width

$11 < 5t$ or 25 in. and < 18 in.

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* A question arises on span lengths: moments in continuous beams and shears in all beams are computed on the clear span, while moments in single spans are figured c/c of bearings. In these tables for simplicity l' (page 122) is taken as the c/c span.

SAFE CARRYING CAPACITY OF SOLID SLABS

Bearing—It is not practicable to tabulate values for bearing on all sorts of wall materials; but, in each case, this should be checked. Safe bearing values are tabulated on page 99. If the wall in this example has a bearing value of 100 psi:—

When $l' = 10'\text{-}0$, $V = 5 \times 278 = 1390$ lb;

$$\text{min bearing} = \frac{1390}{12 \times 100} = 1.16, \text{ say 4 in. min}$$

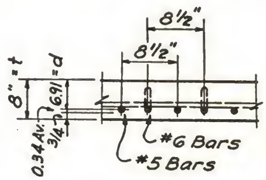
$l' = 15'\text{-}0$, $V = 7.5 \times 123 = 923$ lb;

$$\text{min bearing} = \frac{923}{12 \times 100} = 0.77, \text{ say 4 in. min}$$

Stiffening—Long runs of slab should have the bearings stiffened by a thickened edge beam with a bar at top and a bar at bottom; also, the corners of a building can be reinforced against curling with three or four bars on the bottom diagonally across each corner and with several bars in the top at right angles to the bottom layer extended well back into the floor.

Example II—*End Span, Approximately Balanced Reinforcement*—For the table on page 126, determine the safe carrying capacity on spans of 8 and 16 feet of a slab whose thickness, t, = 8 in.; reinforcement = #5 bars straight alternating with #6 trussed bars, in pairs $8\frac{1}{2}$ in. c/c; weight of slab = 100 psf; steel percentage = 1.28%, and with at least two additional spans $l'' < 1.2\,l' > 0.833\,l'$.

From page 34, for $p = 0.0128$, take $k = 0.394$; $j = 0.869$; $R_c = 230.9$; $R_s = 222.3$. These can also be computed from the formulas on page 23.



$$d = 8 - \tfrac{3}{4} - 0.34 \text{ Av.} = 6.91 \text{ in.}$$

$$p = \frac{(0.31 + 0.44)}{6.91 \times 8\frac{1}{2}} = 0.0128$$

Shear, Continuous End—$V = v_c bjd = 90 \times 12 \times 0.869 \times 6.91 = 6490 \text{ lb} = 1.15\,wl'/2$*

When $l' = 8'\text{-}0''$, $w = \dfrac{6490 \times 2}{1.15 \times 8} = 1410$ psf. Subtracting 100 gives 1310 psf.

$= 16'\text{-}0''$, $w = \dfrac{6490 \times 2}{1.15 \times 16} = 705$ psf. Subtracting 100 gives 605 psf.

Shear, Free End—Since the shear capacity is the same at the free end and the external shear is stated in the Code as $wl'/2$, the continuous end is the determining condition.

Bond, Continuous End—$V = u\Sigma ojd = 300 \times 1.963 \times \dfrac{12}{8.5} \times 0.869 \times 6.91 =$

$$5000 \text{ lb} = 0.70 \times 1.15\,wl'/2\ \dagger$$

When $l' = 8'\text{-}0''$, $w = \dfrac{5000 \times 2}{0.70 \times 1.15 \times 8} = 1550$ psf. Subtracting 100 gives 1450 psf.

$= 16'\text{-}0''$, $w = \dfrac{5000 \times 2}{0.70 \times 1.15 \times 16} = 775$ psf. Subtracting 100 gives 675 psf.

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* The factor 1.15 represents the increase in the shear at the continuous end above that of a symmetrical span (ACI 701c).

† The factor 0.70 represents the percentage of the external end shear that is effective at the point of inflection, which is the maximum shear that needs to be carried by bond on the bottom bars (see page 173). The factor 1.15 represents the increase in the shear at the continuous end above that of a symmetrical span. The product of these two factors can be verified from the middle diagram on page 84 where the $-M = \dfrac{wl^2}{10}$ curve crosses the axis at $0.80l$ so $R_L = 0.40wl$ and $R_R = 0.60wl$; zero shear is at $0.40l$ and at the point of inflection the shear is $\dfrac{0.40}{0.60}\,0.60wl = 0.40wl$ for comparison with $0.70 \times 1.15 \times \dfrac{wl}{2} = 0.403wl$ as used in the computation.

SAFE CARRYING CAPACITY OF SOLID SLABS

Bond, Free End—$V = u\Sigma ojd = 300 \times 1.963 \times \frac{12}{8.5} \times 0.869 \times 6.91 = 5000 \text{ lb} = wl'/2$

When $l' = 8'\text{-}0'', w = \frac{5000 \times 2}{8} = 1250$ psf. Subtracting 100 gives 1150 psf, as given in table on page 126.

$= 16'\text{-}0'', w = \frac{5000 \times 2}{16} = 625$ psf. Subtracting 100 gives 525 psf.

Positive Moment—$M = Rbd^2 = 222.3 \times 12 \times 6.91 \times 6.91 = 127{,}400 \text{ lb-in.} = wl'^2 12/11$

When $l' = 8'\text{-}0'', w = \frac{127{,}400 \times 11}{12 \times 8 \times 8} = 1820$ psf. Subtracting 100 gives 1720 psf.

$= 16'\text{-}0'', w = \frac{127{,}400 \times 11}{12 \times 16 \times 16} = 455$ psf. Subtracting 100 gives 355 psf, as given in table on page 126.

Negative Moment:—At the bottom of the table on page 126, bars are tabulated with this same slab depth for at least two more adjacent interior spans of $l'' = l'$ and for $l'' = 1.2l'$, which is the maximum difference permitted by ACI 318-56.

(a) Check negative moment on a span of 8′-0″ for adjacent span $l'' = l'$, reinforced with #5 bottom and #7 truss bars at 12 in. centers for each pair of bars.

Truss bars, span l'	#6 @ 8.5 = 0.31 sq in./ft
Truss bars, span l''	#7 @ 12 = 0.60
	A_s = 0.91 sq in./ft

$p = \frac{A_s}{bd} = \frac{0.91}{12 \times 6.85} = 0.01107$; $R_s = 194.3$ and $j = 0.876$ (page 34)

When $l'' = l' = 8'\text{-}0''$, max. capacity = 1250 psf (determined by bond on free end); max. $-M = \frac{wl'^2 12}{10} = 1.2 \times 1250 \times 8 \times 8 = 96{,}000$ lb-in.;

$R = \frac{M}{bd^2} = \frac{96{,}000}{12 \times 6.85 \times 6.85} = 170 < 194.3$, so $f_c < 1350$;

$f_s = \frac{M}{A_s jd} = \frac{96{,}000}{0.91 \times 0.876 \times 6.85} = 17{,}600 \text{ psi} < 20{,}000.$

(b) Check negative moment on a span of 8′-0″ for at least two more adjacent spans $l'' = 1.2l' = 9.6$ ft, with #6 bottom and #8 truss bars at 12 in. centers for each pair of bars.

Truss bars, span l'	#6 @ 8.5 = 0.31 sq in./ft
Truss bars, span l''	#8 @ 12 = 0.79
	A_s = 1.10 sq in./ft

$p = \frac{A_s}{bd} = \frac{1.10}{12 \times 6.81} = 0.01346$; $R_s = 233.2$; $k = 0.401$; $j = 0.866$ (page 34)

$$\text{max. } -M = \frac{w}{10}\left[\frac{l' + 1.2l'}{2}\right]^2 12 = \frac{1250}{10}\left[\frac{8 + 9.6}{2}\right]^2 12 = 116{,}200 \text{ lb-in.}$$

$R = \frac{M}{bd^2} = \frac{116{,}200}{12 \times 6.81 \times 6.81} = 209 < 233.2$, so $f_c < 1350$;

$f_s = \frac{M}{A_s jd} = \frac{116{,}200}{1.10 \times 0.866 \times 6.81} = 17{,}900 \text{ psi} < 20{,}000.$

(c) Check negative moment on a span of 16 ft for at least two more adjacent spans $l'' = l' = 16.0$, with #5 bottom and #7 truss bars at 12 in. centers for each pair of bars. As in (a), $p = 0.01107$, $R_s = 194.3$; $j = 0.876$; max. capacity = 455 psf (determined by positive moment); max. $-M = \frac{wl'^2 12}{10} = 1.2 \times 455 \times 16 \times 16 = 140{,}000$ lb-in.

$R = \frac{M}{bd^2} = \frac{140{,}000}{12 \times 6.85 \times 6.85} = 249 > 194.3$, so $f_c > 1350$ psi and compressive steel is required (see page 119). Tensile top steel should be checked for this case, since

SAFE CARRYING CAPACITY OF SOLID SLABS

$R > 194.3;\ A_s = \dfrac{M}{f_s jd} = \dfrac{140{,}000}{20{,}000 \times 0.876 \times 6.85} = 1.168$ sq in./ft

Truss bars as in (a) $= 0.91$

Add top bar for 0.258 sq in./ft

The note in the middle of page 125 recommends 1-#5 top bar at each #6 truss bar, which is #5 @ $8\frac{1}{2} = 0.437$ sq in./ft.

(d) Check negative moment on a span of 16 ft for at least two more adjacent spans $l'' = 1.2l' = 19.2$ ft, with #6 bottom and #8 truss bars at 12 in. centers for each pair of bars. As in (b), $p = 0.01346$, $R_s = 233.2$, $k = 0.401$; $j = 0.866$, with max. capacity = 455 psf, max. $-M = \dfrac{w}{10}\left[\dfrac{l' + 1.2l'}{2}\right]^2 12 = 1.2 \times 455 \times 17.6 \times 17.6 = 169{,}000$ lb-in.;

$R = \dfrac{M}{bd^2} = \dfrac{169{,}000}{12 \times 6.81 \times 6.81} = 303 > 233.2$, so $f_c > 1350$ psi, and compressive steel is needed (see below). The tensile top steel should be checked since $R > 233.2$:—

Required $A_s = \dfrac{M}{f_s jd} = \dfrac{169{,}000}{20{,}000 \times 0.876 \times 6.81} = 1.43$ sq in./ft

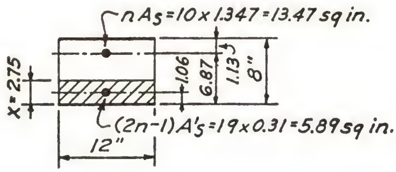
Truss bar as in (c) $= 1.10$

Add top bar for 0.33 sq in./ft

The note in the middle of page 125 recommends 1-#5 top bar at each #6 truss bar, which is #5 @ $8\frac{1}{2} = 0.437$ sq in./ft.

Compressive steel is not required at supports on a span of 8′-0″ (Neg. Mom. (a) and (b), p. 118) but is required on a span of 16′-0″ (Neg. Mom. (c) and (d), p. 118). No great refinement of computation is necessary because either the bottom bars are lapped past each other enough that one set of bottom bars is always available for compression, or both sets of bars are extended through the supports so that two sets of bars are available at the support and extend out only as far as required by the negative moment curve (say 17 bar dia. or $l'/15$ past support).

First consider case (c) under negative moment, where $A_s = 1.347$ sq in. per ft and one set of bottom (compression) bars provides #5 @ 12 in. (span l'') so $A'_s = 0.31$ sq in. per ft.



$$\frac{12x^2}{2} + 5.89 \times 1.06 + 13.47 \times 6.83 = (12x + 5.89 + 13.47)x$$

$$6x^2 + 19.36x = 98.09$$

$$x^2 + 3.227x + (1.614)^2 = 16.35 + 2.60 = 18.95$$

$$x = -1.614 \pm 4.36 = 2.75 \text{ in.}$$

$$C_c = 12 \times 2.75 \times \frac{1350}{2} = 22{,}280. \quad 22{,}280 \times 6.12 = 136{,}400 \text{ lb-in.}$$

$$C_s = 5.89 \times 1350 \times \frac{1.69}{2.75} = 4890. \quad 4{,}890 \times 5.81 = \underline{28{,}400} \text{ ``}$$

Max. $M_R = 164{,}800$ lb-in. $> 140{,}000$ lb-in.

This is well above the 140,000 lb-in. of negative moment computed in (c) and very close to the 169,000 required in (d), while, in this latter case, the bars from l'' are #6 @ 12 in. with a tension area of 1.10 as against 0.91 for (c), so it is hardly necessary to repeat the computations to determine that a simple overlapping splice of the bottom bars will provide sufficient compressive reinforcement for both cases. From the foregoing it can be seen that the amount of compressive steel required increases with the span, and for safety under all possibilities, the distance E_4 on page 125 has been established at 17 bar diameters or $l''/15$ to provide two sets of compressive bars. The user can always make a more precise determination for any individual case.

SAFE CARRYING CAPACITY OF SOLID SLABS

Bond on Top Bars

When two or more bars of differing sizes undergo the same unit stress, the bond varies. The simplest procedure is to apportion the total external shear to any one bar size in the ratio that the a_s provided by that size of bar is to the total A_s and to use for Σo the perimeter of that size of bar. The larger bar is the critical one. Thus for the four cases discussed under Negative Moment on page 118:—

(a) $V = u\Sigma ojd\frac{A_s}{a_s} = 300 \times 2.749\frac{12}{12} \times 0.876 \times 6.85 \times \frac{0.91}{0.60} = 7520 \text{ lb} = 1.15\ w8/2.$
$w = 1635 > 1250$

(b) $V = 300 \times 3.142\frac{12}{12} \times 0.866 \times 6.81 \times \frac{1.10}{0.79} = 7750 = 1.15\,w8/2.$ $w = 1658 > 1250.$

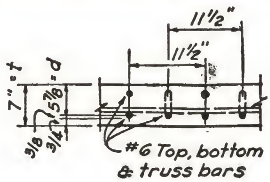
(c) $V = 300 \times 2.749\frac{12}{12}jd\frac{1.347}{0.60} = 11{,}100 = 1.15w16/2.$ $w = 1206 > 455.$

(d) $V = 300 \times 3.142\frac{12}{12}jd\frac{1.537}{0.79} = 10{,}830 = 1.15w16/2.$ $w = 1176 > 455.$

So bond on top bars is not a determining factor in any of these cases.

Temperature Steel, *Bearing*, *Stiffening* } Proceed along the same lines as in Example I.

Example III—*Interior Span, Approximately Balanced Reinforcement*—For the table on page 129, determine the safe carrying capacity on spans of 9 and 18 feet of a slab whose thickness, t, = 7 in.; reinforcement = #6 bottom, #6 truss, #6 top bars, in groups of one each @ 11½ in. c/c; weight of slab = 88 psf; steel percentage = 1.30%; and with at least two additional spans on either end l'' or $l''' < 1.2l' > 0.833\ l'$.



$d = 7 - \frac{3}{4} - \frac{3}{8} = 5\frac{7}{8}$ in.

$p = \frac{0.44}{5\frac{3}{4} \times 5.88} = 0.0130$

From the table on page 34:—
$k = 0.396 \quad j = 0.868 \quad R_s = 225.6$

Shear—$V = v_c bjd = 90 \times 12 \times 0.868 \times 5.88 = 5530 \text{ lb} = wl'/2$

When $l' = 9'\text{-}0'',\ w = \frac{2 \times 5530}{9} = 1230$ psf. Subtracting 88 gives 1142 psf.

$= 18'\text{-}0'',\ w = \frac{2 \times 5530}{18} = 615$ psf. Subtracting 88 gives 527 psf.

Bond (Positive Bars)

$V = u\Sigma ojd = 300 \times 2.356 \times \frac{12}{11\frac{1}{2}} \times 0.868 \times 5.88 = 3770 \text{ lb} = 0.70\,wl'/2$ *

When $l' = 9'\text{-}0'',\ w = \frac{2 \times 3770}{0.70 \times 9} = 1190$ psf. Subtracting 88 gives **1102** psf. As given in the table on page 129.

$= 18'\text{-}0'',\ w = \frac{2 \times 3770}{0.70 \times 18} = 598$ psf. Subtracting 88 gives 510 psf.

Positive Moment

$M = R_s bd^2 = 225.6 \times 12 \times (5.88)^2 = 93{,}700 \text{ lb-in.} = \frac{wl'^2 12}{16}$

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* The factor 0.70 represents the percentage of end shear effective at the point of inflection, where the bond on the bottom bars is a maximum, see footnote on page 173.

SAFE CARRYING CAPACITY OF SOLID SLABS

When $l' = 9'\text{-}0''$, $w = \dfrac{93{,}700 \times 16}{12 \times 9 \times 9} = 1542$ psf. Subtracting 88 gives 1454 psf.

$= 18'\text{-}0''$, $w = \dfrac{93{,}700 \times 16}{12 \times 18 \times 18} = 383$ psf. Subtracting 88 gives **295** psf. As given in the table on page 129.

Negative Moment

(a) When $l'' = l'$:—

$$p = \frac{3 \times 0.44 \times 12}{11\frac{1}{2} \times 12 \times 5.88} = 0.0195; \quad R_s = 330.2 \ \text{(page 34)} \ [R_c = 262.5]$$

$$M_s = R_s bd^2 = 330.2 \times 12 \times (5.88)^2 = 137{,}000 \text{ lb-in.} = \frac{wl'^2 12}{11}*$$

When $l' = 9'\text{-}0''$, $w = \dfrac{137{,}000 \times 11}{12 \times 9 \times 9} = 1550$ psf. Subtracting 88 gives $1462 > 1102$ psf.

$= 18'\text{-}0''$, $w = \dfrac{137{,}000 \times 11}{12 \times 18 \times 18} = 388$ psf. Subtracting 88 gives $300 > 295$ psf.

But when $R_s = 330.2$, $f_c > 1350$ psi (see the table on page 34), so extend bottom bars for compressive reinforcement (see Example II).

(b) When $l'' = 1.2l'$:—

$$M = 137{,}000 \text{ lb-in.} = \frac{w(1.1l')^2\, 12}{11} \text{ (See Example II)}$$

When $l' = 9'\text{-}0''$, $w = \dfrac{137{,}000 \times 11}{1.21 \times 9 \times 9 \times 12} = 1282$ psf. Subtracting 88 gives 1194 psf > 1102 psf.

When $l' = 18'\text{-}0''$, $w = \dfrac{137{,}000 \times 11}{1.21 \times 18 \times 18 \times 12} = 321$ psf. Subtracting 88 gives $233 < 295$ psf.

In those cases where $l'' = 1.2l'$ and where balanced reinforcement is used for positive moment, see page 128 under E_x and E_y and in this case change 1-#6 to 1-#8 top bar.

$$d'/d = \frac{1.125}{5.875} = 0.191$$

Then $p = \dfrac{(2 \times 0.44 + 0.79)}{11\frac{1}{2} \times 5.82} = 0.0249$, and from page 45 $R_s = 425 > 330.2$

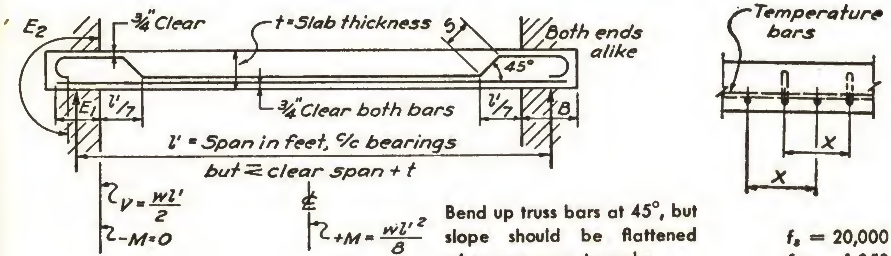
Using twice the actual compressive steel (ACI 706b):—

$$p' = \frac{2 \times 2 \times 0.44}{11\frac{1}{2} \times 5.82} = 0.0264; \quad R' = 360 \pm < 330.2$$

* Though ACI 318-56 permits the use of $-M = wl'^2/12$ for spans of 10 ft or less, for consistency $-wl'^2/11$ was used throughout these tables.

SOLID CONCRETE SLABS—SINGLE SPANS (OR SIMPLE SPANS SIMPLY SUPPORTED)

Applies to the Tables on pages 123 and 124.



Bend up truss bars at 45°, but slope should be flattened when necessary to make $S \ngtr 4\frac{1}{2}$ in.

$f_s = 20{,}000$ psi
$f_c = 1{,}350$ psi
$v_c = 90$ psi
$u = 300$ psi

E_1 = 6 in. minimum for bottom bars.
E_2 = 17 bar diameters (straight if possible, bent if necessary).
B = ordinarily 4 in. minimum but sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall is made (see page 99).
x = distance between consecutive bottom bars = distance between consecutive truss bars = distance between *pairs* of bars.
Alternate bars are trussed as shown. (Sometimes only every fourth bar is trussed on outer end.)

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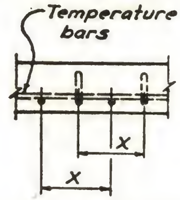
* This embedment theoretically develops the bar on the basis of $L = \frac{f_s d}{4u} = \frac{20{,}000d}{4 \times 300} = 16\frac{2}{3}\,d$. Recent pull-out tests have indicated little or no stress in bars beyond a 10- to 13-diameter embedment.

SOLID CONCRETE SLABS—SINGLE SPANS (OR SIMPLE SPANS SIMPLY SUPPORTED)

Approx. Balanced Reinforcement

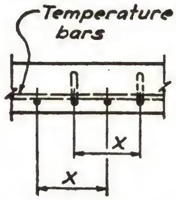


For general instructions and notes on the use of this table, see page 122.

Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom and Truss Bars	#3	#3	#4	#4	#4	#5	#5	#5	#5	#5	#5	#6	#6	#6	#6
Distance "x" (in.)	8	6	10	8	8	10	10	9	8	7	7	9	9	8	8
Steel Percentage	1.33	1.43	1.33	1.43	1.25	1.40	1.26	1.26	1.30	1.37	1.28	1.32	1.24	1.31	1.24
Weight of Slab (psf)	38	44	50	56	63	69	75	81	88	94	100	106	113	119	125
Span	**Safe Superimposed Load (psf)**														
4′-0	436	739	686	1011	1167	1291	1435	1769							
4′-6	348	574	604	897	1028	1131	1268	1554	1932						
5′-0	275	457	539	796	920	1011	1135	1401	1732						
5′-6	220	370	480	720	832	923	1025	1269	1562	1941					
6′-0	179	304	411	595	710	835	935	1142	1432	1796	1920	1894			
6′-6	147	252	342	498	595	765	857	1059	1312	1631	1770	1744	1867		
7′-0	121	211	288	422	505	695	790	977	1211	1504	1635	1609	1727		
7′-6	101	178	244	360	431	597	682	841	1044	1298	1417	1494	1607	1931	
8′-0	84	151	209	310	371	516	590	729	907	1129	1233	1394	1497	1801	1915
8′-6	70	129	179	268	322	449	514	636	793	989	1081	1270	1366	1644	1755
9′-0	58	110	154	233	280	393	451	559	698	873	953	1122	1206	1453	1552
9′-6	48	94	133	203	245	346	397	493	617	773	845	996	1071	1292	1380
10′-0	40	81	116	178	215	305	351	437	548	689	753	888	956	1154	1233
10′-6		69	100	156	189	270	311	389	489	616	674	796	856	1036	1107
11′-0			87	137	167	240	277	347	438	553	605	716	770	933	997
11′-6			75	121	147	214	247	311	393	498	545	646	695	843	902
12′-0			65	106	130	191	220	289	354	449	492	584	629	765	818
12′-6				94	115	170	197	250	319	407	446	530	571	696	744
13′-0				82	101	152	177	225	288	369	405	482	519	634	678
13′-6				72	89	136	158	203	261	335	368	439	473	579	620
14′-0	$\frac{l'}{t} = 32$ Max. Span for Floor Slabs				79	122	142	183	236	305	335	401	432	530	568
14′-6					69	109	127	165	214	278	306	367	395	486	521
15′-0					60	97	114	149	195	253	279	336	362	447	478
15′-6						86	102	134	177	231	255	308	332	411	440
16′-0							91	121	160	211	233	282	304	378	405
16′-6							81	109	145	193	213	259	279	348	373
17′-0							72	98	132	176	195	238	257	321	345
17′-6							64	88	119	161	178	218	236	296	318
18′-0								79	108	147	163	201	217	274	294
18′-6								70	98	134	149	184	199	253	271
19′-0								62	88	122	136	169	183	233	251
19′-6									79	111	124	155	168	215	232
20′-0									71	101	113	142	154	199	214

SOLID CONCRETE SLABS—SINGLE SPANS (OR SIMPLE SPANS SIMPLY SUPPORTED)

Approx. ⅔% Reinforcement



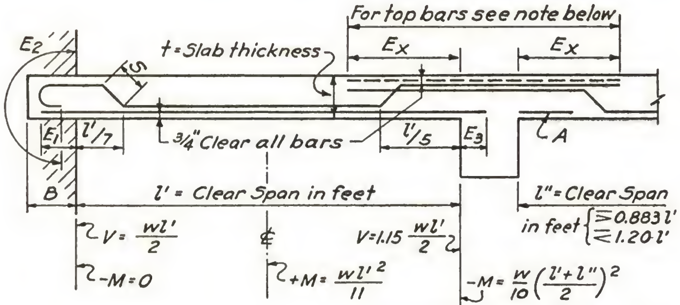
For general instructions and notes on the use of this table, see page 122.

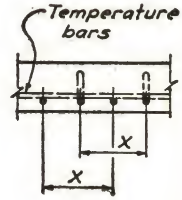
Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom and Truss Bars	#3	#3	#3	#4	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6
Distance "x" (in.)	16	12	11	16	16	12	12	18	16	14	14	18	18	16	16
Steel Percentage	0.667	0.716	0.653	0.714	0.625	0.741	0.667	0.632	0.652	0.687	0.638	0.664	0.622	0.657	0.622
Weight of Slab (psf)	38	44	50	56	63	69	75	81	88	94	100	106	113	119	125
Span	Safe Superimposed Load (psf)														
4'-0	207	362	481	497	572	877	983	877	1092	1364	1478	1452	1557	1871	1985
4'-6	162	287	383	436	502	773	865	770	960	1156	1300	1276	1367	1641	1755
5'-0	124	224	301	386	446	689	772	686	857	1068	1160	1139	1222	1471	1565
5'-6	96	178	240	347	399	619	695	616	772	966	1045	1024	1097	1326	1405
6'-0	75	142	194	292	337	527	590	559	697	877	951	934	999	1206	1282
6'-6	58	115	158	240	278	439	492	510	637	801	870	850	912	1106	1175
7'-0	45	91	129	199	231	369	414	467	587	738	800	784	840	1018	1081
7'-6	34	75	106	166	193	312	351	430	500	633	684	724	778	941	1000
8'-0		61	87	139	162	266	299	340	429	545	589	673	721	873	935
8'-6		49	71	117	136	228	256	292	370	472	510	611	655	797	848
9'-0		38	58	98	115	196	220	251	320	410	444	533	572	698	743
9'-6			47	82	96	168	190	217	278	359	389	468	502	614	654
10'-0			37	69	81	145	164	188	243	315	341	412	442	543	578
10'-6			29	57	67	125	142	163	212	277	300	364	390	481	513
11'-0				47	56	108	123	141	185	244	264	321	345	428	456
11'-6				38	46	93	106	122	164	215	233	285	306	381	407
12'-0				31	37	80	91	106	141	190	206	254	272	340	363
12'-6						68	78	91	123	167	182	225	242	304	325
13'-0						58	66	78	107	148	161	200	215	272	291
13'-6						48	56	66	93	130	142	178	191	244	261
14'-0						40	47	56	80	114	125	157	170	218	233
14'-6						33	39	47	69	100	109	140	151	195	209
15'-0							31	38	59	87	96	123	133	175	187
15'-6									49	76	83	109	118	156	167
16'-0									41	65	72	96	102	139	149
16'-6										56	62	84	90	124	133
17'-0										47	52	73	79	110	118
17'-6										39	44	63	68	97	104
18'-0											36	54	58	85	92
18'-6												45	49	74	80
19'-0												37	40	64	69
19'-6													33	55	60
20'-0														46	50

$\frac{l'}{t} = 32$ {Max. Span for Floor Slabs}

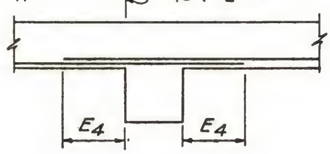
SOLID CONCRETE SLABS—END SPANS

Applies to the Tables on pages 126 and 127.





Bend up truss bars at 45°, but slope should be flattened when necessary to make $S \geqq 4\frac{1}{2}$ in.



f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

E_1 = 6 in. minimum for bottom bars.
E_2 = 17 bar diameters* (straight if possible, bent if necessary).
E_3 = 6 in. minimum with ⅔% reinforcement.
E_4 = not less than $\left\{\begin{matrix}\text{17 bar diameters} \\ l'/15\dagger\end{matrix}\right\}$ with balanced reinforcement.

E_x = to meet ACI 902 (a) extend at least one-third of top bars $l'/3$ and the remainder $l'/6$; if necessary increase extension until bars are anchored 17 dias. past point of max. stress. (Bend-down points.)

Top bar = Truss bars from two sides provide all necessary negative reinforcement over support in these tables when $l'' = l'$. When $l'' = 1.2\,l'$, add one top bar as follows for each truss bar in the end span:—

Truss Bar—#5, add 1-#4 Top.
#6, add 1-#5 Top.
#7, add 1-#5 Top.

B = ordinarily 4 in. minimum and sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall s made (see page 99).
x = distance between consecutive bottom bars = distance between consecutive truss bars = distance between *pairs* of bars.
A = bottom bar in adjoining span, not shown.

The lines marked $l'' = l'$ and $l'' = 1.2\,l'$ at the bottom of the tables give the amount of reinforcing steel required in typical interior (fully-continuous) spans of the same slab thickness and with a span length equal to l' or $1.2\,l'$, respectively, that will provide the same safe carrying capacity as that of the corresponding end span.

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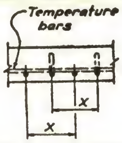
* This embedment theoretically develops the bar on the basis of $L = \frac{f_s d}{4u} = \frac{20{,}000d}{4 \times 300} = 16\frac{2}{3}$. Recent pull-out tests have indicated little or no stress in bars beyond a 10-13-diameter embedment.

† Embedment of bottom bars at interior support is determined by the fact that some of the bottom bars are required for compressive reinforcement. The exact length varies. The maximum is that which will develop the full compression in the bars at the higher unit stress permitted by the ACI Code (706b) and which will at the same time extend the needed distance across the moment curve. The capacity of the slab may be determined by shear, bond or flexure. The recommendation for E_4 will cover the worst condition. The user may at his option work out the needs of any particular problem (see pages 119 and 121).

SOLID CONCRETE SLABS—END SPANS

Approx. Balanced Reinforcement



For general instructions and notes on the use of this table, see page 125.

Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom Bar	#3	#3	#3	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Truss Bar	#4	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7	#7
Distance "x" (in.)	11½*	10	8	11	10	8½*	11½*	11	10	9	8½*	11	10	10	9
Steel Percentage	1.31	1.22	1.29	1.34	1.28	1.34	1.32	1.26	1.27	1.30	1.28	1.28	1.32	1.25	1.31
Weight of Slab (psf)	38½	44	50	56	63	69	75	81	88	94	100	106	113	119	125
Span	**Safe Superimposed Load (psf)**														
4′-0	286	422	643	714	909	1221	1225	1427	1722						
4′-6	248	370	570	629	802	1079	1080	1249	1522						
5′-0	221	328	508	561	715	963	967	1124	1361	1646					
5′-6	198	295	459	504	645	868	871	1015	1227	1488	1705	1676			
6′-0	178	266	419	457	585	791	791	924	1117	1311	1550	1524	1802		
6′-6	161	243	379	419	535	725	725	844	1022	1245	1430	1399	1666		
7′-0	148	220	349	384	492	668	668	780	942	1146	1320	1292	1529	1634	
7′-6	134	204	323	355	455	617	618	724	876	1066	1225	1199	1420	1521	1795
8′-0	123	188	299	330	423	576	575	673	815	994	1150	1119	1326	1413	1675
8′-6	104	162	263	306	394	537	536	629	762	931	1068	1044	1240	1321	1570
9′-0	89	140	230	286	369	503	503	589	716	871	1002	979	1166	1241	1535
9′-6	76	121	200	269	347	475	471	554	672	821	945	924	1097	1173	1390
10′-0	65	105	176	251	322	441	445	521	634	776	892	874	1037	1111	1315
10′-6		91	155	222	286	394	420	493	598	733	845	826	982	1051	1245
11′-0			137	198	255	353	397	468	568	696	802	782	934	996	1185
11′-6			121	176	228	317	377	443	540	661	764	745	887	949	1125
12′-0			107	157	204	285	345	407	499	611	709	709	845	902	1075
12′-6				140	183	257	311	369	453	556	646	677	807	872	1025
13′-0				126	165	233	282	335	412	508	587	648	772	825	985
13′-6				112	148	211	257	304	376	464	539	617	737	730	943
14′-0					133	191	233	277	343	425	494	566	677	729	905
14′-6					120	173	212	253	314	389	455	520	623	670	804
15′-0					108	158	193	231	287	358	418	480	575	621	744
15′-6						143	176	211	264	329	385	443	531	574	688
16′-0							161	193	242	303	355	409	492	530	638
16′-6							147	177	222	279	328	378	455	492	593
17′-0							134	162	204	258	303	350	423	456	551
17′-6							122	148	188	238	280	324	392	424	513
18′-0								136	173	220	259	301	365	394	478
18′-6								124	159	203	240	279	339	367	446
19′-0								113	146	187	222	259	316	341	416
19′-6									134	173	206	240	294	318	389
20′-0									123	160	191	223	274	296	363

In the portion of table above dotted line, live load exceeds three times slab wt (ACI 701c).

$\frac{l'}{t} = 32$ {Max. Span for Floor Slabs

Bottom bars extend through support as per E_4 in figure on page 125

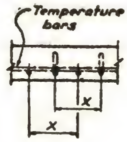
REINFORCEMENT FOR INTERIOR SPAN OF EQUAL CAPACITY

		3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
$l'' = l'$	Bottom Bar	#3	#3	#3	#4	#4	#4	#4	#4	#4	#4	#5	#5	#6	#6	#6
	Truss Bar	#4	#4	#4	#5	#5	#5	#6	#6	#6	#6	#7	#7	#8	#8	#8
	Distance "x" (in.)	16½	14	11	14	10½	10½	13	12½	11½	10	12	10½	14	14	12½
$l'' = 1.2l'$	Bottom Bar	#3	#3	#3	#3	#4	#5	#5	#5	#6	#6	#6	#6	#5	#5	#5
	Truss Bar	#4	#4	#4	#5	#6	#7	#7	#7	#8	#8	#8	#8	#8	#8	#8
	Distance "x" (in.)	11½	9½	7½	9	11½	12½	11	10½	14	12½	12	11	10	10	9

* It is desirable to keep these distances in even inches to suit slab bar chairs.

SOLID CONCRETE SLABS—END SPANS

Approx. ⅔% Reinforcement



For general instructions and notes on the use of this table, see page 125.

Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom Bar	#3	#3	#3	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
Truss Bar	#3	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7	#7
Distance "x" (in.)	16	18	16	22	20	17	23	21	19	18	17	22	20	19	18
Steel Percentage	0.668	0.645	0.640	0.667	0.642	0.671	0.667	0.660	0.669	0.651	0.640	0.643	0.663	0.656	0.653
Weight of Slab (psf)	38	44	50	56	63	69	75	81	88	94	100	106	113	119	125

Span	Safe Superimposed Load (psf)														
4′-0	208	224	311	343	442	599	599	734	890	1036					
4′-6	180	194	271	299	387	525	524	643	780	1006					
5′-0	158	170	239	263	342	466	452	572	693	810					
5′-6	140	140	212	234	305	416	414	510	623	727	835	815	972	1091	1235
6′-0	103	134	191	210	274	377	374	463	563	659	756	740	871	991	1125
6′-6	82	121	172	189	248	342	318	420	513	601	692	674	803	906	1025
7′-0	65	108	157	167	226	312	309	384	470	551	635	619	741	832	945
7′-6	52	98	142	156	207	286	284	354	433	507	585	569	682	770	875
8′-0	41	83	130	143	189	264	262	326	401	471	542	529	632	713	810
8′-6	32	69	111	132	175	246	242	302	372	437	504	492	588	665	755
9′-0		56	93	121	161	228	224	281	343	407	470	458	548	620	706
9′-6		46	79	112	149	212	209	261	323	382	441	428	513	581	662
10′-0		37	66	102	137	196	194	244	303	357	413	400	483	547	624
10′-6			55	88	118	171	183	229	284	336	389	376	451	515	586
11′-0			46	75	102	150	169	215	268	316	366	355	428	485	555
11′-6				64	88	131	158	203	252	298	347	335	405	460	526
12′-0				54	76	115	144	183	232	272	327	316	384	436	499
12′-6				45	65	100	127	162	206	244	286	299	363	424	474
13′-0				38	55	87	112	144	184	218	257	284	343	393	451
13′-6					47	76	98	128	164	195	231	270	328	373	430
14′-0						66	86	113	147	175	208	244	298	342	380
14′-6						57	75	100	131	157	187	220	270	310	355
15′-0							65	88	116	140	168	199	245	282	324
15′-6							56	77	103	125	151	179	222	257	295
16′-0							48	67	91	112	135	162	201	233	269
16′-6								58	81	99	121	146	183	212	246
17′-0								50	71	88	108	131	166	193	224
17′-6								43	62	78	97	118	150	175	204
18′-0									54	68	86	105	135	159	186
18′-6									46	60	76	94	122	144	170
19′-0										52	67	84	110	131	154
19′-6										44	58	74	99	118	140
20′-0										38	50	65	88	106	127

In the portion of table above dotted line, live load exceeds three times slab wt (ACI 701c).

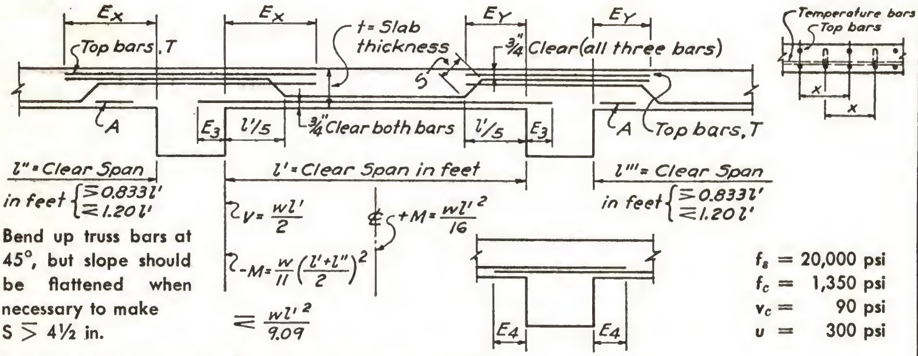
$\frac{l'}{t} = 32$ {Max. Span for Floor Slabs

REINFORCEMENT FOR INTERIOR SPAN OF EQUAL CAPACITY

		3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
$l'' = l'$	Bottom Bar	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5	#5
	Truss Bar	#3	#3	#3	#4	#5	#5	#5	#5	#5	#5	#5	#6	#6	#6	#6
	Distance "x" (in.)	14	15	13	17	16	13	18	17	15	14	14	18	16	15	14
$l'' = 1.2l'$	Bottom Bar	#3	#3	#3	#4	#4	#4	#5	#5	#5	#5	#5	#6	#6	#6	#6
	Truss Bar	#3	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7	#7
	Distance "x" (in.)	10	18	15	18	17	15	18	16	15	14	13	16	15	14	13

SOLID CONCRETE SLABS—INTERIOR SPANS

Applies to the Tables on pages 129 and 130.



Bend up truss bars at 45°, but slope should be flattened when necessary to make $S \geqq 4\frac{1}{2}$ in.

f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

E_3 = 6 in. minimum with ⅔% reinforcement. (Page 130)
E_4 = not less than 17 bar diameters, nor less than $l''/15$,* with balanced reinforcement. (Page 129)

$E_x = E_y =$ {to meet ACI 902 (a) extend at least one-third of the top bars $l'/3$ and remainder at least $l'/6$; if necessary increase extension until bars are anchored 17 dias. past point of max. stress. (Bend-down points)}

Extra top bars over support are required in these tables. When l'' or $l''' = l'$, use bars scheduled in table; when l'' or $l''' = 1.2\ l'$, increase top bars to sizes given below:—

#3 to #4	#6 to #8
#4 to #5	#7 to #9
#5 to #8	

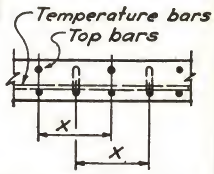
x = distance between consecutive bottom bars = distance between consecutive truss bars = distance between pairs of bars.
T = additional top bars, one over each meeting pair of bottom bars, i.e., one for each set of straight and truss bars.
A = bottom bar in adjoining span, not shown.

---

* Embedment of bottom bars at interior support is determined by the fact that some of the bottom bars are required for compressive reinforcement. The exact length varies. The maximum is that which will develop the full compression in the bars at the higher unit stress permitted by the ACI Code (20,000 psi) and which will at the same time extend the needed distance across the moment curve. The capacity of the slab may be determined by shear, bond or flexure. The recommendation for E_4 will cover the worst condition. The user may at his option work out the needs of any particular problem (see pages 119 and 121).

SOLID CONCRETE SLABS—INTERIOR SPANS

Approx. Balanced Reinforcement



For general instructions and notes on the use of this table, see page 128.

Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom Bar	#3	#4	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7
Truss Bar	#3	#4	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7
Top Bar "T"*	#3	#4	#4	#4	#5	#5	#5	#6	#6	#6	#6	#6	#7	#7	#7
Distance "x" (in.)	8½†	12	10	8½†	12	10½†	9½†	13	11½†	10½†	10	9	12	11	10½†
Steel Percentage	1.25	1.33	1.33	1.33	1.31	1.31	1.31	1.27	1.30	1.30	1.28	1.33	1.28	1.30	1.30
Weight of Slab (psf)	38	44	50	56	63	69	75	81	88	94	100	106	113	119	125
Span	**Safe Superimposed Load (psf)**														
4′-0	608	685	998	1384	1372	1781									
4′-6	528	602	881	1222	1212	1576	1945	1849							
5′-0	472	539	787	1099	1087	1411	1740	1659							
5′-6	425	486	712	989	980	1273	1575	1499	1854						
6′-0	371	442	648	902	893	1163	1437	1369	1692	1914					
6′-6	311	396	594	829	820	1070	1320	1258	1559	1762	1883				
7′-0	263	335	549	765	757	989	1222	1159	1442	1626	1741	1865	1985		
7′-6	224	287	509	710	702	916	1135	1079	1340	1514	1618	1733	1840	1958	
8′-0	192	246	468	649	654	856	1060	1005	1252	1412	1511	1618	1717	1828	1939
8′-6	166	213	408	568	612	801	995	940	1172	1324	1416	1517	1609	1713	1817
9′-0	144	185	358	501	574	752	935	884	1102	1246	1332	1427	1517	1611	1710
9′-6	125	162	317	444	541	710	880	833	1039	1175	1256	1346	1429	1520	1613
10′-0	109	142	281	394	500	646	810	786	982	1112	1189	1273	1352	1438	1526
10′-6		124	251	353	447	580	729	715	932	1053	1127	1208	1282	1364	1447
11′-0			224	317	403	523	658	708	883	1001	1071	1148	1218	1297	1376
11′-6			201	285	363	473	596	674	842	955	1020	1093	1162	1235	1311
12′-0			180	257	329	428	541	624	772	910	974	1043	1107	1179	1251
12′-6				233	298	389	493	569	707	841	931	997	1059	1127	1196
13′-0				211	271	355	450	521	648	772	891	955	1013	1079	1145
13′-6				191	247	324	412	477	594	709	821	916	972	1034	1098
14′-0					225	296	377	438	546	653	756	879	934	993	1054
14′-6					205	272	347	403	503	602	698	845	897	955	1013
15′-0					187	249	319	371	464	557	646	784	845	919	976
15′-6						229	294	342	429	515	599	728	790	886	940
16′-0							271	316	397	478	556	676	734	854	907
16′-6							251	293	368	444	516	630	684	796	875
17′-0							232	271	342	412	481	587	637	743	844
17′-6							214	251	318	384	448	548	595	695	790
18′-0								233	295	358	418	512	556	650	739
18′-6								216	275	334	390	479	521	609	693
19′-0								201	256	311	365	449	488	571	651
19′-6									239	291	341	420	457	536	611
20′-0									223	272	319	394	429	504	575

In the portion of table above dotted line, live load exceeds three times slab wt (ACI 701c).

$\frac{l'}{t} = 32 \left\{ \begin{array}{l} \text{Max. Span} \\ \text{for} \\ \text{Floor Slabs} \end{array} \right.$

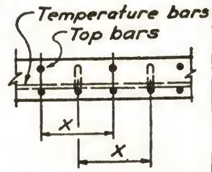
Bottom bars extend through support as per E_4 in figure on page 128.

* See note under E_x and E_y on page 128.

† It is desirable to keep these distances in even inches to suit slab bar chairs.

SOLID CONCRETE SLABS—INTERIOR SPANS

Approx. 2/3% Reinforcement



For general instructions and notes on the use of this table, see page 128.

Slab Thickness (in.)	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10
Temp. Bars	#3	#3	#3	#3	#3	#4	#4	#4	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	15	15½	13½	12	11	18	16½	15	14	13	12½	18	17	16	15½
Bottom Bar	#3	#3	#3	#4	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6
Truss Bar	#3	#3	#3	#4	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6
Top Bar "T"*	#4	#4	#4	#5	#5	#5	#5	#6	#6	#6	#6	#7	#7	#7	#7
Distance "x" (in.)	16	13	11	17	15	13	12	18	16	15	14	18	17	16	15
Steel Percentage	0.668	0.664	0.653	0.666	0.666	0.666	0.666	0.643	0.645	0.646	0.648	0.655	0.656	0.657	0.657
Weight of Slab (psf)	38	44	50	56	63	69	75	81	88	94	100	106	113	119	125
Span	Safe Superimposed Load (psf)														
4′-0	313	493	705	692	901	1191	1435	1284	1597	1856	In the portion of table above				
4′-6	273	433	621	609	792	1051	1267	1131	1407	1636	dotted line, live load exceeds				
5′-0	243	386	554	543	707	936	1135	1011	1262	1466	three times slab wt (ACI 701c).				
5′-6	217	347	499	488	637	845	1025	911	1147	1326	1535	1504	1717	1951	
6′-0	188	302	438	442	579	768	935	829	1034	1206	1400	1369	1566	1781	
6′-6	154	252	366	404	528	704	855	758	950	1106	1285	1256	1437	1629	1855
7′-0	128	211	309	371	487	648	789	699	874	1021	1185	1159	1327	1501	1715
7′-6	106	178	262	342	450	601	731	646	812	946	1100	1075	1229	1396	1605
8′-0	89	151	224	310	415	535	672	602	755	881	1025	1004	1149	1301	1474
8′-6	74	129	193	268	360	467	586	563	704	823	960	936	1072	1220	1385
9′-0	62	110	167	233	315	409	516	525	660	771	900	879	1007	1142	1305
9′-6	52	94	145	203	276	360	455	493	622	727	846	826	947	1076	1227
10′-0	43	80	125	178	242	317	402	464	575	675	796	781	895	1016	1160
10′-6		69	109	156	214	282	359	415	504	604	712	738	847	963	1100
11′-0			95	136	190	251	320	370	453	543	640	699	803	914	1045
11′-6			83	121	168	224	287	333	407	489	579	667	762	868	995
12′-0			72	106	149	200	257	299	367	441	523	604	697	799	904
12′-6				94	136	179	231	269	331	399	475	549	635	728	824
13′-0				82	118	160	208	243	299	362	431	499	578	664	754
13′-6					105	143	187	219	271	329	393	455	528	607	691
14′-0					93	128	169	198	246	299	358	416	483	556	633
14′-6						115	152	179	223	272	327	380	443	510	582
15′-0						103	137	162	203	248	299	348	406	469	535
15′-6							124	147	184	227	274	320	373	432	494
16′-0							112	133	168	207	250	293	343	398	455
16′-6							100	120	152	189	230	270	316	367	421
17′-0							90	108	138	172	210	248	291	339	389
17′-6								97	126	157	193	228	268	313	360
18′-0								88	114	144	177	209	247	289	334
18′-6									103	131	162	193	228	268	309
19′-0									93	119	148	177	210	247	286
19′-6										108	136	163	194	229	266
20′-0										98	124	149	179	212	246

$\frac{l'}{t} = 32 \left\{ \text{Max. Span for Floor Slabs} \right.$

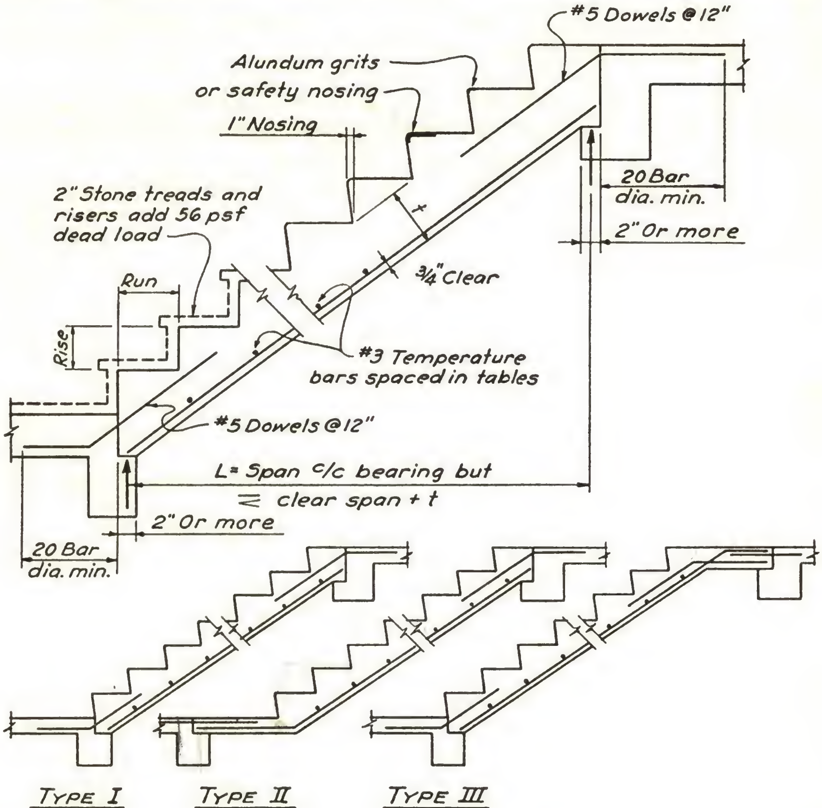
* See note under E_x and E_y on page 128.

STAIR SLABS—SINGLE SPANS

There are many possible combinations of rise and run, surface finishes and landing arrangements.

Risers and runs vary from 6-on-12 to 8-on-9. Surfaces vary from troweling alundum grits into the finished concrete to adding asphalt tile or linoleum, and on up to 1½ or 2 inches of terrazzo, stone or marble.

Landings may be a part of the floor construction (Type I) or an integral portion of the bottom of the stair (Type II) or built integrally at the top of the flight (Type III). Landing slabs might have their thickness reduced below that of the flight, being at a point where the bending moment is less. This refinement is only occasionally used and must be checked for strength. For Type II, the reinforcing steel is bent around the corner as in the diagram. For Type III, the bars should be bent and lapped around the re-entrant corner as shown, otherwise they would pull out. They are lapped a minimum of 20 bar diameters past each other and anchored in the top part of the slab. Landings are sometimes spanned crosswise of the flight to shorten the span of the stair slab.



STAIR SLABS—SINGLE SPANS

Stair slabs are properly designed as horizontal spans, using the horizontally-projected load per square foot, the horizontal projection of the clear span and the inclined depth from the heel of the step to the soffit of the stair slab. Stairs are usually poured after the main structure, resting in pockets in the supporting beams and doweled to them. While the dowels might develop restraint, it is customary to design stair slabs as single spans without continuity.

Safe carrying capacities were obtained by computing the least safe total load as determined by shear or bond or flexure, and by deducting the weight of the slab itself (including the triangular step) to obtain the safe superimposed load. The tabulated capacity includes live load, finishes, ceilings, balustrades, partitions and everything but the dead weight of the structural concrete. Live loads are usually from 75 to 100 psf and, on main staircases, as much as 125 psf.

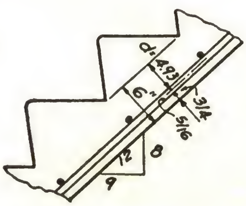
Tables are included for stair slabs on spans from 5 to 14 feet (7 to 18 risers), with rise-to-run ratios of 8-to-9 and 7-to-10½, for one set of stresses, viz., $f_s = 20{,}000$ psi, $f_c = 1350$ psi, and for the one case of single spans, as continuity is ordinarily not a factor in stair design.

For convenience, these tables may be entered in two ways:—(1) the horizontal projection L may be used directly; (2) if only the story height is known, it can be subdivided into a number of equal risers and the number of risers used for entering the table.

Example—For page 134, determine the capacity of a 6-inch stair slab reinforced with #5 bars, 5 in. c/c, on a span of 12′-0″ (corresponding to about 16 risers of 8-on-9 steps).

$$p = \frac{A_s \text{ (sq in. per 12 in.)}}{db} = \frac{0.31 \times 12}{5 \times 4.93 \times 12} = 0.0126$$

Solution:-



Resisting Moment: (See page 34), Max. Allowable $R_s = 219$

$$M = Rbd^2 = 219 \times 12 \times \overline{4.93}^2 = w \times \overline{12}^2 \times \frac{12}{8}$$

$$w = 296 \text{ psf (total)}$$

Weight of Slab:—

$$6 \times \frac{12}{9} @ 12\frac{1}{2} = 100$$

$$4 \times 12\frac{1}{2} = \underline{50^*} \quad 150 \text{ psf (dead weight)}$$

$$\overline{146} \text{ psf (as given in the table on page 134)}$$

Shear—$V = bdjv_c = 4.93 \times 12 \times \frac{7}{8} \times 90 = \frac{w\,12}{2}$

$$w = 776 \text{ psf}$$

Bond—$V = \Sigma ojdu = 1.963 \times \frac{12}{5} \times \frac{7}{8} \times 4.93 \times 300 = \frac{w\,12}{2}$

$$w = 1016 \text{ psf}$$

Temperature Bars

$A_s = 0.002 \times 6.00 \times 12 = 0.144$

#3 @ 9 = 0.146

* The series of 8″ steps (8 x 9 triangles) is equivalent to a 4″ horizontal slab.

RISE AND RUN OF STAIRS

Many rules have been proposed for the proper relation of rise to run, such as, rise + run = 17, rise × run = 75, and 2 × rise + run = 25; but a somewhat more comfortable stair results from the values in the following table:—

Rise	Run	Rise	Run	Rise	Run
6	12⅝	7	10¾	Following Are Extra Steep	
6⅛	12⅜	7⅛	10⅝		
6¼	12⅛	7¼	10⅜	7⅞	9½
6⅜	11⅞	7⅜	10¼	8	9¼
6½	11⅝	7½	10	8⅛	9⅛
6⅝	11½	7⅝	9⅞	8¼	9
6¾	11¼	7¾	9⅝	8⅜	8¾
6⅞	11			8½	8⅝

Stairs are usually limited to not less than 3 rises (because of the danger of tripping on one or two steps) and not more than about 18 rises in one flight between landings. The table below gives the vertical height for any number of rises and can be used for the length of run by taking one less run than the number of rises in the flight:—

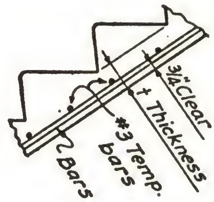
TOTAL HEIGHT (OR LENGTH) (FT AND IN.)

Rise or Run (in.)	Number of Rises (or Runs)															
	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
6	1-6	2-0	2-6	3-0	3-6	4-0	4-6	5-0	5-6	6-0	6-6	7-0	7-6	8-0	8-6	9-0
6¼	1-6¾	2-1	2-7¼	3-1½	3-7¾	4-2	4-8¼	5-2½	5-8¾	6-3	6-9¼	7-3½	7-9¾	8-4	8-10¼	9-4½
6½	1-7½	2-2	2-8½	3-3	3-9½	4-4	4-10½	5-5	5-11½	6-6	7-0½	7-7	8-1½	8-8	9-2½	9-9
6¾	1-8¼	2-3	2-9¾	3-4½	3-11¼	4-6	5-0¾	5-7½	6-2¼	6-9	7-3¾	7-10½	8-5¼	9-0	9-6¾	10-1½
7	1-9	2-4	2-11	3-6	4-1	4-8	5-3	5-10	6-5	7-0	7-7	8-2	8-9	9-4	9-11	10-6
7¼	1-9¾	2-5	3-0¼	3-7½	4-2¾	4-10	5-5¼	6-0½	6-7¾	7-3	7-10¼	8-5½	9-0¾	9-8	10-3¼	10-10½
7½	1-10½	2-6	3-1½	3-9	4-4½	5-0	5-7½	6-3	6-10½	7-6	8-1½	8-9	9-4½	10-0	10-7½	11-3
7¾	1-11¼	2-7	3-2¾	3-10½	4-6¼	5-2	5-9¾	6-5½	7-1¼	7-9	8-4¾	9-0½	9-8¼	10-4	10-11¾	11-7½
8	2-0	2-8	3-4	4-0	4-8	5-4	6-0	6-8	7-4	8-0	8-8	9-4	10-0	10-8	11-4	12-0
8¼	2-0¾	2-9	3-5¼	4-1½	4-9¾	5-6	6-2¼	6-10½	7-6¾	8-3	8-11¼	9-7½	10-3¾	11-0	11-8¼	12-4½
8½	2-1½	2-10	3-6½	4-3	4-11½	5-8	6-4½	7-1	7-9½	8-6	9-2½	9-11	10-7½	11-4	12-0½	12-9
8¾	2-2¼	2-11	3-7¾	4-4½	5-1¼	5-10	6-6¾	7-3½	8-0¼	8-9	9-5¾	10-2½	10-11¼	11-8	12-4¾	13-1½
9	2-3	3-0	3-9	4-6	5-3	6-0	6-9	7-6	8-3	9-0	9-9	10-6	11-3	12-0	12-9	13-6
9¼	2-3¾	3-1	3-10¼	4-7½	5-4¾	6-2	6-11¼	7-8½	8-5¾	9-3	10-0¼	10-9½	11-6¾	12-4	13-1¼	13-10½
9½	2-4½	3-2	3-11½	4-9	5-6½	6-4	7-1½	7-11	8-8½	9-6	10-3½	11-1	11-10½	12-8	13-5½	14-3
9¾	2-5¼	3-3	4-0¾	4-10½	5-8¼	6-6	7-3¾	8-1½	8-11¼	9-9	10-6¾	11-4½	12-2¼	13-0	13-9¾	14-7½
10	2-6	3-4	4-2	5-0	5-10	6-8	7-6	8-4	9-2	10-0	10-10	11-8	12-6	13-4	14-2	15-0
10¼	2-6¾	3-5	4-3¼	5-1½	5-11¾	6-10	7-8¼	8-6½	9-4¾	10-3	11-1¼	11-11½	12-9¾	13-8	14-6¼	15-4½
10½	2-7½	3-6	4-4½	5-3	6-1½	7-0	7-10½	8-9	9-7½	10-6	11-4½	12-3	13-1½	14-0	14-10½	15-9
10¾	2-8¼	3-7	4-5¾	5-4½	6-3¼	7-2	8-0¾	8-11½	9-10¼	10-9	11-7¾	12-6½	13-5¼	14-4	15-2¾	16-1½
11	2-9	3-8	4-7	5-6	6-5	7-4	8-3	9-2	10-1	11-0	11-11	12-10	13-9	14-8	15-7	16-6
11¼	2-9¾	3-9	4-8¼	5-7½	6-6¾	7-6	8-5¼	9-4½	10-3¾	11-3	12-2¼	13-1½	14-0¾	15-0	15-11¼	16-10½
11½	2-10½	3-10	4-9½	5-9	6-8½	7-8	8-7½	9-7	10-6½	11-6	12-5½	13-5	14-4½	15-4	16-3½	17-3
11¾	2-11¼	3-11	4-10¾	5-10½	6-10¼	7-10	8-9¾	9-9½	10-9¼	11-9	12-8¾	13-8½	14-8¼	15-8	16-7¾	17-7½
12	3-0	4-0	5-0	6-0	7-0	8-0	9-0	10-0	11-0	12-0	13-0	14-0	15-0	16-0	17-0	18-0

STAIR SLABS—SINGLE SPANS

9 in. Run, 8 in. Rise

For description of use of table, see pages 131 ff.



Approx. Balanced Reinforcement ($p = 0.0136$)

$f_s = 20{,}000$ psi $\quad f_c = 1350$ psi $\quad v_c = 90$ psi $\quad u = 300$ psi

Safe superimposed load in pounds per square foot includes weight of any plastered ceiling, finished treads and finished risers. (Weight of *all* concrete of stair slab and steps has already been deducted.)

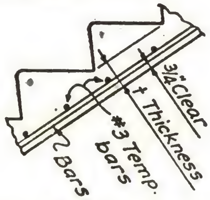
Slab Thickness (t) (in.)		3	3½	4	4½	5	5½	6	6½	7
Bars		#3	#3	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)		4	3	5	4	4	5	5	4½	4
Spacing of #3 Temp. Bars (in.)		15	15½	13½	12	11	10	9	8½	8
Weight of Concrete (psf hor. proj.)		101	109	117	125	134	142	150	158	167
Approx. No. of Risers *	Span (L) Horizontal	Safe Superimposed Load (psf)								
7	5′-0	215								
7	5′-6	161								
8	6′-0	119	236							
8	6′-6	86	185							
9	7′-0	60	144	222						
10	7′-6		112	178						
10	8′-0		85	138	237					
11	8′-6		63	113	196					
12	9′-0			88	161	212				
12	9′-6			67	132	176				
13	10′-6				107	146				
14	10′-6				85	120	207	239		
14	11′-0				66	100	176	205		
15	11′-6					80	149	174		
16	12′-0					61	126	146	206	
16	12′-6						105	125	178	214
17	13′-0						86	104	152	187
18	13′-6						69	85	129	161
18	14′-0							69	109	138

* The rise of stairs may vary from about 6 in. to 8 in. and the run from about 11 in. to 9 in.

STAIR SLABS—SINGLE SPANS

10½ in. Run, 7 in. Rise

For description of use of table, see pages 131 ff.



Approx. Balanced Reinforcement ($p = 0.0136$)

$f_s = 20{,}000$ psi $f_c = 1350$ psi $v_c = 90$ psi $u = 300$ psi

Safe superimposed load in pounds per square foot includes weight of any plastered ceiling, finished treads and finished risers. (Weight of *all* concrete of stair slab and steps has already been deducted.)

Slab Thickness (t) (in.)	3	3½	4	4½	5	5½	6	6½	7
Bars	#3	#3	#4	#4	#4	#5	#5	#5	#5
Spacing (in.)	4	3	5	4	4	5	5	4½	4
Spacing of #3 Temp. Bars	15	15½	13½	12	11	10	9	8½	8
Weight of Concrete (psf hor. proj.)	91	98	105	112	120	128	135	142	150

Approx. No. of Risers *	Span (L) Horizontal	Safe Superimposed Load (psf)								
5	5′-0	225								
6	5′-6	171								
6	6′-0	129	247							
7	6′-6	96	196							
8	7′-0	70	155	234						
8	7′-6	59	123	190						
9	8′-0		96	150	250					
9	8′-6		74	125	209					
10	9′-0		55	100	174	226				
10	9′-6			79	145	190				
11	10′-0			61	120	160				
12	10′-6				98	134	221	254		
12	11′-0				79	114	190	220		
13	11′-6				63	94	163	289		
14	12′-0					75	140	163	222	
14	12′-6					57	119	140	194	231
15	13′-0						100	119	168	204
15	13′-6						83	100	145	178
16	14′-0						68	84	125	155
16	14′-6							69	105	133
17	15′-0								86	112
18	15′-6									92

* The rise of stairs may vary from about 6 in. to 8 in. and the run from about 11 in. to 9 in.

CONCRETE JOIST CONSTRUCTION

Concrete joist construction consists of narrow ribs or joists and a top slab of concrete, the whole formed by creating longitudinal void spaces by means of permanent or removable forms of steel or removable forms of wood. Joist widths vary from 4 to 7 or 8 inches. Standard forms for the void spaces are usually 20 in. or 30 in. wide and have a depth of 6, 8, 10, 12 or 14 in. The top slab is usually 2, 2½ or 3 in. thick, but not less than $\frac{1}{12}$ of the clear distance between ribs.

The following tables give the safe superimposed load in pounds per square foot (psf), i.e., the total carrying capacity as determined by the least of various factors such as shear, bond or flexure, with only the dead weight of the concrete deducted. Thus the safe superimposed load includes live load, partition allowance, floor finishes, fills, ceilings and everything but the dead weight of the concrete construction.

Since the tables are rather elaborate, a general outline of what is covered may be helpful. The tables are divided into three sections:—(1) single spans, (2) end spans, and (3) interior spans. Each of these sections has a short explanation, schedule of limitations, sketch of the recommended requirements and an illustrative example. Each section is subdivided into two parts, the first for 20-in. wide forms and the second for 30-in. wide forms. Within each such section, the tables are divided by depth of form, and finally various practicable thicknesses of top slab are given for each form depth.

All tables are based upon the recommendations of the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," and in the case of end spans the positive moment is here taken as $wl'^2/11$, i.e., without restraint at the outer end. Bond and diagonal tension values are based upon deformed bars conforming to ASTM A305. Attention is directed to the fact that plain bars or deformed bars not meeting A305 cannot be used with these tables.

The arrangement of bars and chairs, and the bending, spacing, lapping and embedment of bars are all in accordance with "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

The load capacities are accurate for the particular conditions described, but there are so many variables in loads, stiffnesses, continuity and in the quality of materials that the services of a structural engineer are recommended for final designs of important structures.

CONCRETE JOIST CONSTRUCTION

TEMPERATURE REINFORCEMENT

Temperature reinforcement in the concrete top slab over forms and in a direction normal to the span of the joists shall be of bars or welded wire fabric at least equal in area to the values in the following table and increased where necessary for flexure, giving due consideration to concentrated loads:—

Thickness of Top Slab	Bars	Welded Wire Fabric
2 in.	#2* @ 10	6 in. mesh 8 ga. wire
2½ "	#2* @ 8	6 " " 6 " " or 4 in. mesh 8 ga. wire
3 "	#2* @ 6½ or #3 @ 14	6 in. mesh 5 ga. wire
	All tied with #2 @ 4′-2″ c/c	

* Plain round bars.

TAPERED ENDS

Conforming to U. S. Department of Commerce Simplified Practice Recommendation R87-32, tapered end forms are available which increase the effective joist width 2 in. on each side for 20 in. wide forms and 2½ in. on each side for 30 in. wide forms in a distance of 3 ft from the end. Above and to the right of the zigzag line in the tables, tapered ends must be used or the tabulated values proportionately reduced. Below and to the left of these lines, square (nontapered) ends are adequate.

DEFORMATION WITH TIME

It is recommended that the maximum span for floor construction, particularly where partitions extend parallel to the joists, should be limited to 24 times total depth of construction (t). This limitation is indicated by a horizontal line across each table. This ratio is retained for single, end and interior conditions. For roofs, or where time-sagging is not important, span/depth ratios may exceed 24, if desired.

DISTRIBUTING RIBS

For floor construction, use distributing ribs with at least 1-#4 bar top and 1-#4 bar bottom as follows:—

One in the center of spans from 20 ft to 30 ft and
two at the third points of spans over 30 ft.

STRESSES

As noted at the head of each set of tables, steel is stressed to 20,000 psi. Where it is necessary to use some other stress, vary the steel areas in direct proportion. Concrete is assumed to test 3000 psi, and n is taken as 10. If weaker concrete is used, the capacity should be reduced accordingly. The capacity can be increased with stronger concrete.

UNEQUAL CONTINUOUS SPANS

Bending moments are computed from the clear span for positive moment in continuous spans and from the average of the two adjacent clear spans for negative moment. The assumptions are made that the larger of two adjacent

CONCRETE JOIST CONSTRUCTION

spans does not exceed the shorter by more than 20 per cent and that the unit live load does not exceed three times the unit dead load. For cases outside of these limitations, the moments must be corrected by more accurate methods (see pages 66-80).

The values tabulated under End Spans apply accurately only when there are two additional approximately equal spans continuous with the one under consideration. If there is only one adjacent span, the negative moment should be $wl'^2/9$ instead of $wl'^2/10$, and those values that are governed by negative moment would be reduced accordingly. Values obtained from positive moment would not change.

The values tabulated under Interior Spans apply accurately only when there are two additional approximately equal spans continuous with each end of the one under consideration. If either end is continuous with an end span, the negative moment should be $wl'^2/10$ instead of $wl'^2/11$ and the safe load as governed by negative moment should be reduced accordingly. Values obtained from positive moment would not change. This condition can also be checked by reference to the same data in the tables for End Spans.

LIVE LOAD LIMITATIONS

ACI 318-56-701(c) establishes moment factors for cases where the unit live load does not exceed three times the unit dead load. If the ratio is greater, the effect of unbalanced panel loads may require more accurate analysis (see pages 66-80).

The dead load in such analyses includes not only the weight of the concrete slab (which is here deducted from the total load to obtain the safe superimposed load), but any ceilings, floor finishes, partitions, and similar immovable features. Hence it is not practicable to indicate in these tables the points where the unit live load is exactly equal to three times the unit dead load. The safe superimposed loads have been tabulated to values somewhat above 300 psf and then stopped. The user is cautioned to check the ratio of live to dead loads in the higher capacities.

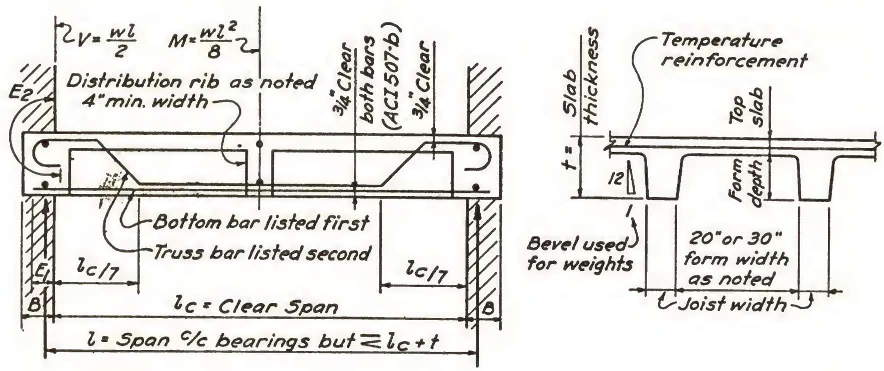
CONCRETE JOIST CONSTRUCTION

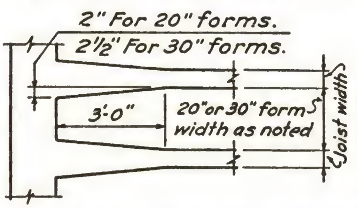
SINGLE SPAN

The general description of concrete joist construction on pages 136 to 138 should be read in connection with these explanations.

Tables on pages 142 to 154, inclusive, give the safe superimposed loads on single (noncontinuous) spans of concrete joist construction. For continuous spans, see the table on pages 158 to 186.

The arrangement of joists and of reinforcing bars should be as shown on the figure below:—





PLAN OF STANDARD TAPERED END WHERE REQUIRED.

STRESSES:—
f'_c = 3000 psi
f_c = 1350 psi
f_s = 20,000 psi
v = 90 psi *
u = 300 psi *

CODES:—
"Building Code Requirements for Reinforced Concrete (ACI 318-56)"; "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

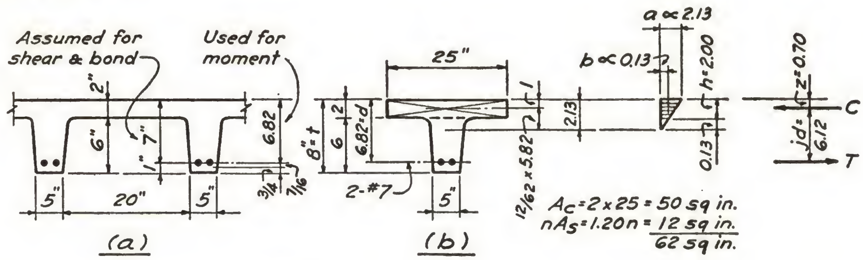
E_1 = 6 in. minimum for bottom bars.
E_2 = 17 bar diameters (24 diameters, when $d > 12$ in.), usually requiring a semicircular hook.
B = ordinarily 4 in. minimum (or preferably 6 in.) and sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall is made (see page 99). Most designers prefer to have a continuous distribution rib of concrete bearing on the wall and reinforced with at least 1-#4 bar top and 1-#4 bar bottom whenever it is desired to spread the load along the wall bearing. When providing bearing on existing masonry walls, it is often sufficient merely to extend the joist stems into individual pockets cut into the wall, but it is then recommended that the stiffening rib be placed between the joists parallel and adjacent to the wall bearing.

Almost all usual combinations of form depth, top slab and joist reinforcement are presented herewith. To show how the tables for single spans were computed and to permit extension beyond the scope of the tables if required, an illustrative example is shown on the following page:—

* Bond and diagonal tension values are based upon deformed bars meeting ASTM A305. Plain round bars or deformed bars not meeting ASTM A305 will not give sufficient bond resistance.

CONCRETE JOIST CONSTRUCTION

Example—Determine the safe carrying capacity on spans of 12 and 18 ft of 6-in. deep forms plus 2-in. of top slab with 5 in. wide joists at 25 in. centers and reinforced with 1-#7 straight bar and 1-#7 bent bar. Dead weight of the concrete construction is 42 psf.* (See page 142.)



Live load capacity determined by:—

(a) *Shear*

Max. Allowable $V = vbjd = 90 \times 5 \times \frac{7}{8} \times 7 = 2756$ lb $= 2.08\, wl/2$

For $l = 12$; $w = 221$ psf. Subtracting 42 gives 179 psf.
For $l = 18$; $w = 147$ psf. Subtracting 42 gives 105 psf.

(b) *Bond*

Max. Allowable $V = u\Sigma ojd = 300 \times 2.749 \times \frac{7}{8} \times 7.0 = 5050$ lb $= 2.08\, wl/2$

For $l = 12$; $w = 405$ psf. Subtracting 42 gives 363 psf.
For $l = 18$; $w = 270$ psf. Subtracting 42 gives 228 psf.

(c) *Positive Moment* $t/d = 2/6.82 = 0.294$

$$p = \frac{1.20}{25 \times 6.82} = 0.00704$$

Since this is less than balanced reinforcement, the table on page 35 will not apply, although the charts on pages 40-41 could be used.

Taking moments about slab center in the figure above and neglecting any small compression in the stem

$$kd = 1 + \frac{12}{62} \times 5.82 = 2.13 \text{ in. (or just below the flange)}$$

$$k = \frac{2.13}{6.82} = 0.312 \text{ (see also chart on page 39)}$$

$$z = \text{c. of g. of trapezoid} = \frac{h}{3}\left(\frac{a+2b}{a+b}\right) = \frac{2}{3}\left(\frac{2.13+0.26}{2.13+0.13}\right) = 0.70 \text{ in.}$$

$$jd = 6.82 - 0.70 = 6.12 \text{ in.} \quad j = 0.901 \text{ (see also chart on page 39)}$$

Max. Allowable $M = f_sA_sjd = 20{,}000 \times 1.20 \times 6.12 = 147{,}000$ lb-in. $= 2.08\dfrac{wl^2 12}{8}$

For $l = 12$; $w = 326$ psf. Subtracting 42 gives 284 psf. } *In table, p. 142*
For $l = 18$; $w = 145$ psf. Subtracting 42 gives 103 psf. }

The 12 ft span can carry the capacity load as determined by moment without overstress in bond but with excessively high shear intensity so that computation of the shear capacity of a tapered end is required.

(d) *Tapered End Shear*

Max. Allowable $V = v_cbjd = 90 \times 9 \times \frac{7}{8} \times 7 = 4961$ lb $= 2.08\, wl/2$

For $l = 12$; $w = 397$ psf. Subtracting 42 gives 355 psf.
For $l = 18$; $w = 265$ psf. Subtracting 42 gives 223 psf.

---

* Dead weight is computed:—

2 in. top slab $= 25$ psf

Joist: $\dfrac{5 + 6.0}{2} \times \dfrac{6 \times 150}{2.08 \times 144} = \underline{17}$ psf

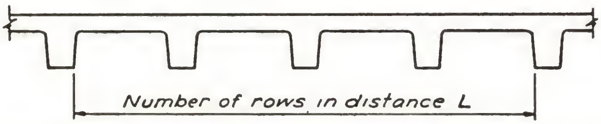
42 psf

CONCRETE JOIST CONSTRUCTION

Summary—On a span of 18 ft the capacity is limited to 103 psf by tension in the reinforcing steel resisting positive moment, the compression in the concrete is less than the allowable (since $p = 0.00704$), and the bond (at 228 psf) and shear on a nontapered end (105 psf) are both adequate.

On a span of 12 ft the capacity by moment is 284 psf, the bond at 363 psf is more than adequate, but a tapered end (355 psf) is required as a nontapered end (179 psf) would be overstressed in diagonal tension.

CONCRETE JOIST SPACING



No. Rows	No. 5″ Joists	Pan Width 19½″	Pan Width 19¾″	Pan Width 20″	Pan Width 20¼″	Pan Width 20½″
1	—	1′-7½	1′-7¾	1′-8	1′-8¼	1′-8½
2	1	3′-8	3′-8½	3′-9	3′-9½	3′-10
3	2	5′-8½	5′-9¼	5′-10	5′-10¾	5′-11½
4	3	7′-9	7′-10	7′-11	8′-0	8′-1
5	4	9′-9½	9′-10¾	10′-0	10′-1¼	10′-2½
6	5	11′-10	11′-11½	12′-1	12′-2½	12′-4
7	6	13′-10½	14′-0¼	14′-2	14′-3¾	14′-5½
8	7	15′-11	16′-1	16′-3	16′-5	16′-7
9	8	17′-11½	18′-1¾	18′-4	18′-6¼	18′-8½
10	9	20′-0	20′-2½	20′-5	20′-7½	20′-10
11	10	22′-0½	22′-3¼	22′-6	22′-8¾	22′-11½
12	11	24′-1	24′-4	24′-7	24′-10	25′-1
13	12	26′-1½	26′-4¾	26′-8	26′-11¼	27′-2½
14	13	28′-2	28′-5½	28′-9	29′-0½	29′-4
15	14	30′-2½	30′-6¼	30′-10	31′-1¾	31′-5½
16	15	32′-3	32′-7	32′-11	33′-3	33′-7
17	16	34′-3½	34′-7¾	35′-0	35′-4¼	35′-8½
18	17	36′-4	36′-8½	37′-1	37′-5½	37′-10
19	18	38′-4½	38′-9¼	39′-2	39′-6¾	39′-11½
20	19	40′-5	40′-10	41′-3	41′-8	42′-1
21	20	42′-5½	42′-10¾	43′-4	43′-9¼	44′-2½
22	21	44′-6	44′-11½	45′-5	45′-10½	46′-4
23	22	46′-6½	47′-0¼	47′-6	47′-11¾	48′-5½

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139 to 141.

Depth	6″ FORMS + 2″ CONCRETE									
Joists	4″ Joists @ 24″ c/c Wt 39 psf					5″ Joists @ 25″ c/c Wt 42 psf				
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Length of Span in Feet										
9	177	233	288	352		271	334			
10	136	181	226	278	328	212	261	310	370	
11	105	143	180	223	263	168	209	248	298	344
12	82	114	145	181	215	134	169	202	244	284
13	64	91	118	149	178	108	138	166	201	236
14	50	73	96	123	148	87	113	137	168	197
15	39	59	79	102	124	70	92	114	140	166
16		47	65	85	104	57	76	95	118	141
17		37	53	71	88	46	63	79	100	120
18			43	59	74	36	51	66	84	103

Depth	6″ FORMS + 2½″ CONCRETE									
Joists	4″ Joists @ 24″ c/c Wt 45 psf					5″ Joists @ 25″ c/c Wt 48 psf				
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Length of Span in Feet										
9	186	246	306	376		289	355			
10	142	191	239	295	350	224	278	331		
11	109	150	190	236	282	177	221	266	318	
12	85	119	152	191	229	141	179	215	258	302
13	65	94	123	156	188	112	145	176	213	249
14	50	75	100	129	156	91	119	145	177	210
15	38	60	81	106	130	73	97	120	148	176
16		47	66	88	109	58	79	99	124	149
17		36	53	74	91	46	64	82	104	126
18			42	60	77	35	53	69	88	108
19			33	49	64		42	57	74	91

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 6″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 52 psf					5″ Joists @ 25″ c/c Wt 55 psf				
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Length of Span in Feet										
9	195	258	323			305	378			
10	148	200	251	313	371	236	295	350		
11	113	156	198	248	298	185	233	281	336	
12	87	123	158	201	241	147	187	226	273	321
13	66	97	127	163	198	117	151	184	225	265
14	50	76	103	134	163	93	123	151	186	221
15	37	60	83	110	136	74	100	125	155	185
16		46	66	90	113	58	81	103	130	156
17		35	53	74	94	45	65	85	108	132
18			41	60	78	34	52	70	90	112
19			32	49	65		42	57	76	95
20				39	53		32	46	63	80

Depth: 8″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 45 psf					5″ Joists @ 25″ c/c Wt 48 psf						
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Length of Span in Feet												
10	181	239	298	365		280	346					
11	141	189	238	294	350	223	277	331				
12	112	152	192	240	287	180	225	271	324			
13	88	123	157	198	238	146	185	224	269	317		
14	70	100	129	164	199	119	152	186	225	266	313	
15	55	81	107	137	167	97	127	156	190	226	266	308
16	43	66	88	115	141	80	106	131	161	192	228	264
17	33	53	73	97	121	65	88	110	137	165	196	228
18		42	60	81	102	53	73	93	117	142	170	199
19		34	49	68	87	43	61	79	100	123	147	174
20			40	57	74	34	50	66	86	106	129	152
21			32	48	63		41	56	73	91	112	133
22				40	53		33	46	63	79	98	117
23				32	45			39	53	68	86	103

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth	8" FORMS + 2½" CONCRETE											
Joists	4" Joists @ 24" c/c Wt 51 psf					5" Joists @ 25" c/c Wt 54 psf						
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Length of Span in Feet 10	187	251	312			295	363					
11	146	198	249	308		234	290	346				
12	114	158	201	250	300	188	235	283	340			
13	90	127	164	206	248	152	192	233	282	330		
14	70	102	134	170	207	124	158	194	235	276	325	
15	55	83	110	142	174	101	131	162	198	234	277	320
16	42	66	91	118	146	82	109	135	168	199	237	274
17	31	53	74	99	124	66	90	114	142	170	203	237
18		42	61	83	105	53	74	95	121	147	176	206
19		32	49	69	89	43	61	80	103	125	152	178
20			40	57	75	33	50	67	87	108	132	156
21			31	47	63		40	56	74	92	115	136
22				38	53		32	46	63	80	100	119
23				31	44			37	53	69	87	104

Depth	8" FORMS + 3" CONCRETE											
Joists	4" Joists @ 24" c/c Wt 58 psf					5" Joists @ 25" c/c Wt 61 psf						
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Length of Span in Feet 10	194	259	325			307						
11	150	204	258	320		243	302	362				
12	116	162	208	260	313	194	245	295	355			
13	91	130	168	212	258	156	199	242	293	344		
14	70	104	137	175	214	127	163	201	245	289	339	
15	54	83	112	145	179	103	134	166	205	243	288	333
16	40	66	91	121	150	82	111	139	173	207	246	286
17		52	74	100	126	66	91	116	146	175	211	245
18		40	60	83	107	52	74	97	124	151	182	213
19		30	48	68	90	41	60	81	105	129	157	185
20			37	56	75	30	49	67	89	110	135	161
21				45	63		38	55	75	94	117	140
22				36	52		30	45	63	81	102	122
23					43			36	52	68	88	106

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 10″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 50 psf					5″ Joists @ 25″ c/c Wt 54 psf							
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Length of Span in Feet													
15	73	106	137	176	213	126	162	198	242	286	338		
16	58	87	114	148	181	104	136	168	205	246	290	336	
17	46	71	96	125	154	87	114	142	176	211	251	293	337
18	35	58	80	106	132	72	96	121	151	182	218	254	294
19		47	66	90	113	58	80	102	130	158	190	222	259
20		37	55	76	98	47	67	88	112	138	166	195	228
21			45	65	84	38	56	74	96	119	146	172	201
22			37	55	72	31	46	63	83	105	128	152	180
23				46	62		38	53	72	91	113	134	160
24				38	52		30	44	61	79	99	119	142
25					44			36	52	69	87	105	126
26					37			30	44	59	76	93	113
27									37	51	67	83	101
28									31	43	58	73	90

Depth: 10″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 56 psf					5″ Joists @ 25″ c/c Wt 60 psf							
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Length of Span in Feet													
15	73	107	140	180	218	128	166	203	248	294	346		
16	57	87	116	151	185	105	139	171	212	250	297	344	
17	44	71	97	127	157	87	116	145	181	214	256	297	
18	33	57	80	108	135	71	97	123	154	185	222	259	301
19		45	66	91	115	58	81	104	132	160	193	227	263
20		36	54	76	98	46	67	88	113	138	169	198	232
21			44	64	84	36	56	74	97	120	147	174	205
22			35	54	72		45	63	83	104	129	153	181
23				44	61		37	52	71	90	113	135	161
24				36	51			42	60	78	98	120	143
25					43			35	51	67	86	105	126
26					35				42	57	75	93	113
27									35	49	65	82	100
28										41	57	72	88

Values below horizontal line are for spans in excess of 24 f.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 63 psf					5″ Joists @ 25″ c/c Wt 67 psf							
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Length of Span in Feet													
15	71	107	142	184	223	130	170	208	256	302			
16	55	86	117	154	189	106	142	175	217	258	307		
17	41	69	97	129	160	87	118	148	184	220	264	307	
18	30	55	79	108	136	70	98	124	157	189	229	267	309
19		43	65	91	116	56	81	105	134	163	199	233	270
20		32	52	76	98	44	66	88	114	140	173	203	237
21			41	63	83	33	54	74	98	121	150	178	207
22			32	52	70		43	61	84	105	130	156	185
23				42	59		34	50	70	90	114	137	162
24				33	49			40	59	77	98	120	143
25					40			32	49	65	85	105	129
26					32				40	55	73	93	112
27									32	47	63	81	99
28										38	54	70	87

Depth: 12″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 57 psf			5″ Joists @ 25″ c/c Wt 61 psf						6″ Joists @ 26″ c/c Wt 66 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Length of Span in Feet													
18	98	129	160	147	185	223	266	309		290	336		
19	82	110	138	126	159	194	232	271	314	253	294	334	
20	68	94	119	108	138	169	203	239	277	222	259	295	
21	57	80	102	92	119	147	179	211	246	195	229	262	303
22	47	68	88	78	103	129	158	187	218	172	202	233	270
23	38	57	76	66	89	112	139	166	194	152	180	207	241
24	30	47	65	56	77	98	123	147	173	134	159	184	216
25		39	55	47	66	86	108	131	155	118	142	165	194
26		32	47	39	56	74	95	116	139	104	126	147	174
27			39	31	48	65	84	103	124	92	112	132	157
28			32		40	56	74	92	111	81	100	118	141
29					34	48	64	81	100	71	88	105	127
30						41	56	72	89	62	78	94	115
31		No Tapered Ends Required				34	49	63	79	54	69	84	103
32							42	56	71	46	61	75	92
33							36	49	63	40	53	67	83

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 12″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 63 psf			5″ Joists @ 25″ c/c Wt 67 psf						6″ Joists @ 26″ c/c Wt 72 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Length of Span in Feet													
18	98	130	162	149	188	225	269	315		294	341		
19	81	110	139	127	161	195	235	275	319	257	299		
20	67	93	119	108	139	169	205	241	282	224	263	300	
21	55	79	102	92	120	148	180	213	249	197	232	266	309
22	45	66	88	78	103	129	158	188	221	173	205	236	275
23	35	55	75	65	89	112	139	167	196	153	180	209	246
24		45	64	55	76	97	122	147	175	133	160	186	219
25		37	54	45	65	84	107	130	156	118	142	166	197
26			45	36	55	73	94	116	139	103	126	148	177
27			37		46	62	82	102	124	90	111	132	159
28			30		38	53	72	91	110	79	98	117	142
29					31	45	62	80	98	67	87	104	128
30	No Tapered Ends Required					38	54	70	88	60	77	93	114
31						31	46	61	78	51	67	82	103
32							39	53	69	43	59	73	92
33							33	46	61	37	51	65	82

Depth: 12″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 69 psf			5″ Joists @ 25″ c/c Wt 74 psf						6″ Joists @ 26″ c/c Wt 78 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Length of Span in Feet													
18	98	131	165	150	190	227	273	320		300			
19	81	111	141	127	162	196	238	279	325	261	306		
20	66	93	120	108	139	170	207	244	286	228	268	305	
21	53	78	102	90	119	148	181	215	252	199	236	269	316
22	43	65	87	76	102	128	158	189	223	175	208	239	281
23	33	54	74	63	87	111	138	167	198	153	184	212	250
24		44	62	52	74	95	121	147	176	134	162	189	223
25		35	52	42	62	82	106	129	155	117	142	167	200
26			43	33	52	70	92	114	139	103	127	148	179
27			35		43	60	80	100	124	90	112	132	161
28					35	51	69	88	109	78	98	117	144
29						42	60	77	96	67	86	104	128
30						34	51	67	85	58	74	93	114
31	No Tapered Ends Required						43	58	75	49	65	81	102
32							36	50	66	41	57	72	92
33								42	57	34	48	63	81

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 14″ FORMS + 2″ CONCRETE

Joists	5″ Joists @ 25″ c/c Wt 68 psf							6″ Joists @ 26″ c/c Wt 73 psf				
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet: 21	85	111	143	174	212	250	290	232	271	313		
22	71	94	124	152	187	221	259	205	241	278	323	
23	59	80	108	134	166	196	230	181	214	249	289	332
24	49	69	93	117	146	175	207	160	192	222	259	301
25	40	58	80	103	130	156	185	142	170	199	233	270
26	31	48	69	90	115	139	166	126	152	178	210	244
27		40	59	79	101	124	148	112	135	161	190	221
28		32	50	68	89	110	133	98	120	144	171	200
29			42	59	79	98	119	87	107	129	154	181
30			35	51	69	87	107	76	96	116	140	165
31				43	60	77	96	66	85	104	126	150
32				36	52	69	86	58	75	93	114	136
33				30	45	60	77	50	67	83	103	124
34					38	53	69	43	58	74	93	113
35						46	61	37	51	66	83	102
36						40	54	30	44	58	75	92
37						34	47		38	51	67	83
38							41		32	44	59	75

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 25″ c/c Wt 75 psf							6″ Joists @ 26″ c/c Wt 80 psf				
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet: 21	83	109	142	173	212	250	293	232	274	313		
22	69	92	122	152	187	221	260	204	242	278	324	
23	57	78	105	133	164	196	231	181	214	248	290	
24	46	66	90	115	145	174	206	159	190	221	260	300
25	36	55	78	100	127	154	184	141	169	198	234	270
26		45	66	87	112	136	165	123	151	176	209	244
27		36	56	76	98	122	147	109	133	158	188	220
28			45	64	86	108	132	96	119	141	170	199
29			38	55	75	95	117	84	105	126	152	180
30			31	47	65	84	105	73	93	113	137	163
31				39	56	74	94	63	82	100	124	147
32				32	48	65	83	54	72	89	110	134
33					41	56	74	46	63	80	99	121
34					34	49	65	39	53	70	89	110
35						42	57	32	45	61	79	98
36						36	50		38	54	70	89
37							44		32	47	62	80
38							37			40	55	71

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 14″ FORMS + 3″ CONCRETE

Joists	5″ Joists @ 25″ c/c Wt 81 psf							6″ Joists @ 26″ c/c Wt 86 psf				
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet												
21	81	108	141	174	213	253	295	235	275	316		
22	67	92	122	152	187	223	262	206	244	281	328	
23	54	77	104	132	164	197	232	182	215	250	294	
24	43	64	89	115	144	174	207	160	191	222	262	304
25	33	52	76	99	127	155	184	140	169	198	235	273
26		42	64	86	111	136	165	123	150	176	210	246
27		33	53	73	97	121	147	108	133	157	189	222
28			44	63	84	106	130	94	118	140	170	199
29			35	53	73	93	116	82	104	125	153	180
30				44	63	82	103	71	91	111	136	162
31				36	54	72	91	61	80	98	122	146
32					45	62	80	52	69	87	110	132
33					38	54	71	44	60	77	98	120
34					31	46	63	36	52	67	87	108
35						39	54		44	59	77	96
36						33	47		37	51	68	86
37							40		30	43	60	77
38							33			36	52	69

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 6″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 41 psf					5″ Joists @ 35″ c/c Wt 43 psf					6″ Joists @ 36″ c/c Wt 45 psf	
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#6	#7
Length of Span in Feet												
9	122	165	206	256	286	197	245	275	327		315	315
10	91	125	160	200	238	152	190	227	272	291	262	279
11	68	97	125	158	189	118	150	180	218	255	208	245
12	51	75	98	126	153	92	119	145	176	207	168	198
13	37	57	77	101	124	72	95	117	143	170	136	162
14		44	61	82	101	56	76	95	118	141	111	134
15		33	48	66	83	43	61	77	97	117	91	111
16			37	53	68	33	48	62	80	98	75	92
17				43	55		38	50	66	81	61	76
18				33	45			40	54	68	49	63
19					36			32	44	56	40	52

Values below horizontal line are for spans in excess of 24 *t*.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 6″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 48 psf					5″ Joists @ 35″ c/c Wt 50 psf					6″ Joists @ 36″ c/c Wt 52 psf	
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#6	#7
Length of Span in Feet												
9	126	171	217	269	302	207	258	290	347		335	335
10	93	130	167	210	251	158	200	240	289	307	277	294
11	68	99	129	164	199	121	156	190	229	269	220	259
12	50	75	100	131	159	94	123	151	185	218	176	209
13	35	57	79	104	128	73	97	121	150	178	143	170
14		43	61	83	104	56	77	97	122	147	115	140
15		31	47	66	85	42	61	79	100	121	94	115
16			35	53	68	31	48	63	82	101	76	95
17				41	55		36	50	66	83	61	78
18				31	44			39	54	69	49	64
19					34			30	43	57	39	52
20									34	46	30	42

Depth: 8″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 45 psf					5″ Joists @ 35″ c/c Wt 48 psf						6″ Joists @ 36″ c/c Wt 50 psf		
Bottom Bar	#4	#5	#5	#6	#6	#6	#6	#7	#7	#8	#8	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#6	#6	#7	#7	#8	#7	#7	#8
Length of Span in Feet														
10	123	168	212	262	313	250	300	357						
11	94	131	167	208	250	198	239	287	332			319		
12	72	102	133	168	203	158	193	233	274	318		262	308	343
13	54	81	107	136	168	128	157	192	226	266	307	216	255	295
14	40	63	86	111	137	103	128	158	188	223	258	180	214	247
15	30	49	69	91	114	84	106	132	158	188	218	150	180	209
16		38	55	74	94	68	87	110	132	160	186	126	152	178
17			43	61	78	54	72	92	112	136	160	106	129	152
18			34	49	65	43	59	77	95	116	137	89	109	130
19				39	54	34	48	64	80	99	118	74	93	111
20				31	44		38	53	67	85	102	62	79	96
21					36		30	43	57	72	88	52	67	82
22								35	48	62	76	43	57	70
23									39	52	65	35	48	60

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 8″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 51 psf					5″ Joists @ 35″ c/c Wt 54 psf						6″ Joists @ 36″ c/c Wt 56 psf		
Bottom Bar	#4	#5	#5	#6	#6	#6	#6	#7	#7	#8	#8	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#6	#6	#7	#7	#8	#7	#7	#8
Length of Span in Feet														
10	127	173	219	272	325	259	313							
11	96	134	172	217	260	205	248	300	350			340		
12	72	105	137	174	211	164	200	243	286	336		274	324	
13	54	82	109	140	172	131	162	198	236	278	319	225	267	308
14	40	63	87	114	141	106	132	164	196	232	269	187	223	258
15		48	69	92	116	85	108	136	163	195	227	155	186	217
16		36	54	75	96	68	89	113	137	165	193	130	157	185
17			42	60	79	54	72	94	115	140	165	108	133	156
18			32	48	65	42	59	78	97	119	141	91	113	134
19				38	53	32	47	64	81	101	121	75	95	115
20				30	43		37	53	68	86	104	62	80	98
21					34			43	56	73	89	51	67	83
22								34	47	62	77	42	57	71
23									38	52	65	34	47	60

Depth: 10″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 50 psf				5″ Joists @ 35″ c/c Wt 52 psf						6″ Joists @ 36″ c/c Wt 55 psf				
Bottom Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#4	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet															
15	65	89	117	144	136	168	200	238	276	318	265	305			
16	51	72	96	120	113	142	170	203	236	273	226	261	300	307	307
17	39	58	79	101	94	119	144	174	203	236	193	225	259	272	272
18	30	46	66	85	79	101	123	149	176	205	166	196	226	246	246
19		36	54	71	65	85	105	129	152	178	144	169	197	223	223
20			43	59	54	72	90	111	132	156	124	148	172	201	203
21			35	49	44	60	76	96	115	137	108	129	151	177	186
22				40	35	50	65	83	100	120	93	112	133	156	170
23				32		41	55	71	87	105	81	98	117	138	157
24						33	46	61	76	93	70	86	102	123	143
25							38	52	66	81	59	74	90	109	127
26							32	44	57	71	51	65	79	96	114
27								37	49	62	43	56	69	85	101
28								31	42	54	36	48	61	76	91

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 56 psf				5″ Joists @ 35″ c/c Wt 58 psf						6″ Joists @ 36″ c/c Wt 61 psf				
Bottom Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#4	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet															
15	64	89	118	146	138	172	204	245	284	302	271	314			
16	49	71	97	122	115	144	173	209	242	266	231	269	309	316	316
17	37	57	80	102	95	121	146	178	208	236	197	231	266	283	283
18		44	65	84	78	102	124	153	181	210	170	200	231	253	253
19		34	52	70	64	85	106	131	156	182	147	173	201	219	229
20			42	58	52	71	90	113	134	158	126	149	176	205	208
21			33	47	42	59	76	96	117	138	109	130	153	180	191
22				38	33	48	64	82	101	121	94	113	135	159	175
23				30		40	54	71	88	105	81	98	118	140	161
24						31	44	59	76	92	69	85	103	124	145
25							36	50	65	81	59	75	90	109	128
26								42	56	69	50	63	79	96	114
27								35	47	60	42	54	69	85	101
28									40	52	34	46	59	75	90

Depth: 12″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 54 psf		5″ Joists @ 35″ c/c Wt 57 psf						6″ Joists @ 36″ c/c Wt 61 psf				
Bottom Bar	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet													
18	83	105	97	125	151	183	215	250	204	238	271	292	
19	68	89	81	106	130	159	187	219	176	207	237	266	266
20	56	75	68	90	111	137	163	191	153	181	207	242	242
21	46	63	56	76	96	119	142	169	133	158	183	214	222
22	37	52	46	64	82	104	125	148	116	139	161	190	204
23		43	37	54	71	90	110	131	101	121	142	168	189
24		35	30	45	60	78	96	115	87	107	125	150	175
25				37	51	67	84	102	76	94	111	133	157
26				30	42	58	73	90	66	82	97	119	141
27					35	50	64	79	56	72	86	106	126
28						42	55	69	48	62	76	94	113
29						35	46	61	40	54	66	83	101
30						30	41	53	34	46	58	74	90
31	No Tapered Ends Required						34	46		39	50	65	80
32								40		33	44	57	72
33								34			38	50	64

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-141.

Depth: 12″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 60 psf		5″ Joists @ 35″ c/c Wt 63 psf						6″ Joists @ 36″ c/c Wt 67 psf				
Bottom Bar	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Length of Span in Feet													
18	82	105	97	125	152	185	219	252	207	241	276	300	300
19	67	88	80	105	130	159	189	222	178	210	241	272	272
20	55	74	66	89	111	137	164	194	154	183	210	247	247
21	44	61	54	75	95	119	143	171	134	160	184	217	226
22	35	50	44	63	81	103	125	149	116	140	162	192	208
23		41	35	52	68	88	109	131	101	122	142	170	192
24		33		42	58	76	95	115	87	107	126	151	177
25				34	48	65	83	100	75	92	110	134	157
26					40	55	71	89	63	81	97	119	140
27					32	47	62	78	54	70	85	105	126
28						39	52	67	45	60	74	93	112
29						32	45	58	38	51	64	82	100
30							38	50	31	43	56	72	89
31	No Tapered Ends Required						32	43		36	48	63	79
32								37			41	55	70
33								31			35	48	62

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 35″ c/c Wt 62 psf					6″ Joists @ 36″ c/c Wt 66 psf				7″ Joists @ 37″ c/c Wt 70 psf				
Bottom Bar	#7	#7	#8	#8	#9	#9	#9	#10	#10	#9	#10	#10	#11	#11
Truss Bar	#6	#7	#7	#8	#8	#8	#9	#9	#10	#9	#9	#10	#10	#11
Length of Span in Feet														
21	93	115	142	170	200	189	218	254	292	206	242	278	300	
22	79	100	125	149	177	167	193	226	261	182	214	248	277	277
23	67	86	109	131	157	147	171	201	232	160	190	220	256	256
24	56	74	95	115	139	129	151	179	208	141	169	197	229	238
25	47	63	82	102	123	114	135	160	186	125	150	176	206	221
26	38	54	71	89	109	100	119	143	168	110	133	157	185	197
27	31	45	62	78	96	88	106	128	150	97	118	140	166	189
28		37	53	68	85	77	94	114	135	86	105	126	150	171
29		31	45	59	75	67	82	102	121	75	93	112	135	154
30			38	51	66	59	73	91	110	65	83	101	121	139
31			32	44	58	51	64	81	98	57	73	90	109	126
32				38	50	43	56	71	88	49	64	80	98	114
33				32	44	37	49	63	79	42	56	71	88	103
34					37	30	42	56	71	35	48	63	79	93
35					31		36	49	63		42	55	71	84
36							30	42	56		35	48	63	75
37								37	49		30	42	56	68
38								31	43			36	49	60

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
SINGLE SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

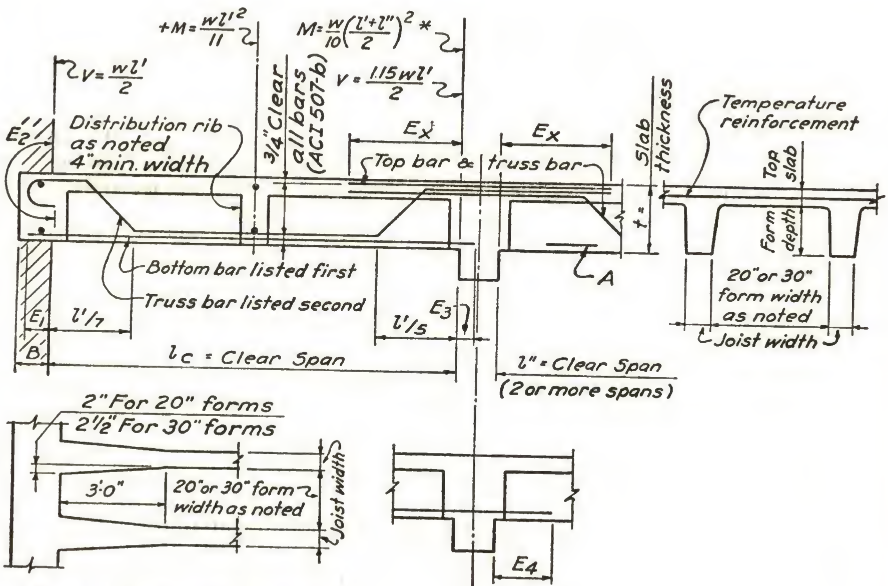
For limitations and explanation of use of tables, see pages 139-141.

Depth	14″ FORMS + 3″ CONCRETE													
Joists	5″ Joists @ 35″ c/c Wt 68 psf					6″ Joists @ 36″ c/c Wt 72 psf				7″ Joists @ 37″ c/c Wt 76 psf				
Bottom Bar	#7	#7	#8	#8	#9	#9	#9	#10	#10	#9	#10	#10	#11	#11
Truss Bar	#6	#7	#7	#8	#8	#8	#9	#9	#10	#9	#9	#10	#10	#11
Length of Span in Feet														
21	91	114	142	170	200	189	219	257	263	207	244	281	304	
22	77	98	123	148	177	166	193	227	243	182	215	250	280	280
23	64	84	107	130	155	145	170	202	224	160	190	222	256	259
24	53	72	93	114	138	128	150	179	208	140	168	198	228	241
25	44	61	80	100	121	112	133	160	187	124	150	177	205	224
26	36	51	69	87	107	99	117	142	168	108	132	157	184	210
27		42	59	76	94	86	104	127	149	95	117	139	165	188
28		34	50	66	83	75	92	113	134	83	104	124	148	170
29			42	56	73	65	80	100	120	72	91	111	132	153
30			35	49	63	56	70	88	107	62	80	98	119	138
31				41	55	48	61	78	96	54	70	87	106	124
32				34	47	40	53	69	85	46	62	77	95	112
33					41	34	46	61	76	38	53	68	85	101
34					34		39	53	68	32	46	60	75	90
35							32	46	59		39	52	67	81
36								39	52		32	45	59	73
37								34	46			39	52	64
38									40			33	45	57

Values below horizontal line are for spans in excess of 24 t.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION—END SPAN

Read the general explanation of the arrangement of tables for Concrete Joist Construction on pages 136 to 138 before using these tables for end spans. The details of temperature reinforcement, tapered end forms, distribution ribs, and especially the type of deformed bars all apply equally well here.



PLAN OF STANDARD TAPERED END WHERE REQUIRED

STRESSES:—

f'_c = 3000 psi
f_c = 1350 psi
f_s = 20,000 psi
v_c = 90 psi †
u = 300 psi †

CODES:—"Building Code Requirements for Reinforced Concrete (ACI 318-56)"; "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

E_1 = 6 in. minimum for bottom bars.

E_2 = 17 bar diameters, (24 dia. for $d > 12''$), (obtained by straight embedment if possible, bent if necessary).

* These tables provide for a negative moment $M = wl'^2/10$ at the continuous end, which is applicable when there is *more than one* additional span beyond the interior support. In the case of a structure only two spans wide, this negative moment should be $M = wl'^2/9$. This would require increasing the total negative reinforcement by approximately 10 per cent, which is best done by increasing the extra top bar. It is also recommended that the bottom bars be extended distance E_4 in this case.

† Bond and diagonal tension values are based upon deformed bars meeting ASTM A305. Plain round bars or deformed bars not meeting ASTM A305 will not give sufficient bond resistance.

CONCRETE JOIST CONSTRUCTION—END SPAN

E_3 = bottom bar to extend 6 in. into the support except when values in the load tables are printed in boldface type.

E_4 = When the values in the load tables are printed in boldface type, bottom bar should extend not less than 17 bar diameters nor less than $l''/10$ * past the far face of the support.

E_x = not less than $\left\{\begin{array}{l} l'/4 \\ l''/4 \\ \text{17 bar diameters} \\ \text{past bend-down} \\ \text{point (24 dia.} \\ \text{when } d > 12 \text{ in.).} \end{array}\right\}$ whichever is greatest. (ACI 902(a) requires top bars to extend to $l'/16$, d, or half bond length past point of inflection.)

The top bar in the table is scheduled on the basis of the adjoining span providing a bent bar of area equal to that of the bent bar in the span under consideration; any considerable variation in negative moment by reason of changes in load, span length, or end restraint of the adjacent span must be worked out by the general principles of continuity (pages 66-81).

B = ordinarily 4 in. minimum (or preferably 6 in.) and sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall is made (see page 99). Most designers prefer to have a continuous distribution rib of concrete bearing on the wall and reinforced with at least one #4 bar top and one #4 bar bottom whenever it is desired to spread the load along the wall bearing. When providing bearing on existing masonry work, it is often sufficient merely to extend the joist stems into individual pockets cut into the wall, but it is then recommended that the stiffening rib be placed between the joists parallel and adjacent to the wall bearing.

A = bottom bar in adjoining span, not shown.

Almost all usual combinations of form depth, top slab and reinforcement are presented herewith. To show how the tables for end spans were computed and to permit extension of the tables if required, an illustrative example is shown:—

Example—Determine the safe carrying capacity on spans of 16 and 21 feet of 8 in. deep forms plus 2½ in. of top slab with 5 in. wide joists at 25 in. centers and reinforced with one #6 bottom bar, one #6 truss bar and one #4 top bar, as shown in the figure on the following page. (See page 160.)

Solution:-

The dead weight of the slab can be computed as on page 140 to be 54 psf.

Shear Max. allowable $V = v_c bjd = 90 \times 5 \times \frac{7}{8} \times 9.38 = 3700$ lb;

$$2.08\frac{wl' \times 1.15\dagger}{2} = 3700 \text{ lb}$$

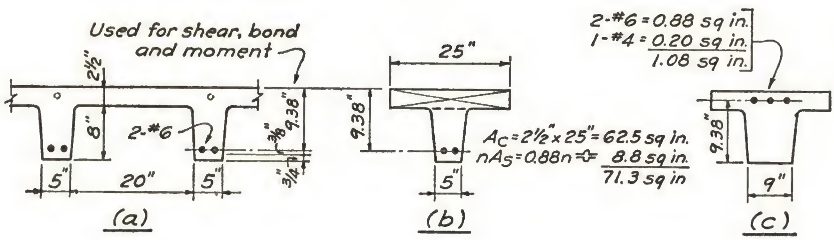
For $l' = 16$; $w = 193$ psf. Subtracting 54 gives 139 psf.
For $l' = 21$; $w = 147$ psf. Subtracting 54 gives 93 psf.
} Use tapered ends to increase shear capacity

---

* Embedment of bottom bar at interior support is determined by the fact that the bottom bar is required for compressive reinforcement. The exact length varies. The maximum is that which will develop the full compression in the bar at the higher unit stress permitted by the ACI Code (20,000 psi) (ACI 318-56, Art. 706-b) and which will at the same time extend the needed distance across the moment curve. The capacity of the joist may be determined by shear, bond or flexure. The recommendation for E_4 will cover the worst condition. The user may at his option work out the needs of any particular problem (see page 173).

† Shear is increased at the continuous end as per ACI Code 701c.

CONCRETE JOIST CONSTRUCTION—END SPAN



Bond Max. allowable $V = u\Sigma ojd = 300 \times 2.356 \times \frac{7}{8} \times 9.38 = 5810$ lb.

$$2.08\frac{wl'}{2} = 5810; \qquad w = \frac{5586}{l'}$$

For $l' = 16$; $w = 349$ psf. Subtracting 54 gives 295 psf.
For $l' = 21$; $w = 266$ psf. Subtracting 54 gives 212 psf.

Positive Moment $A_s = 2\text{-}\#6 = 0.88$ sq in.
Since $t = 2.5$ in., it is likely that the neutral axis lies within the flange; if so:—

$$p = \frac{0.88}{25 \times 9.38} = 0.00375 < 0.0136 \text{ (underreinforced)}$$

from page 34, $k = 0.238$; $kd = 0.238 \times 9.38 = 2.23 < 2.5$ in.
from page 34, $j = 0.920$

Max. allowable $M_s = A_s f_s jd = 0.88 \times 20{,}000 \times 0.920 \times 9.38 = 152{,}000$ lb-in $=$

$$2.08 \times \frac{wl'^2 \times 12}{11}$$

For $l' = 16$; $w = 261$ psf. Subtracting 54 gives 207 psf.
For $l' = 21$; $w = 152$ psf. Subtracting 54 gives 98 psf. } *In table, p. 160*

Tapered End Max. allowable $V = v_c bjd = 90 \times 9 \times \frac{7}{8} \times 9.38 = 6640$ lb;

$$2.08 \times \frac{wl' \times 1.15}{2} = 6640 \text{ lb}$$

Shear For $l' = 16$; $w = 347$ psf. Subtracting 54 gives 293 psf.
For $l' = 21$; $w = 265$ psf. Subtracting 54 gives 211 psf.

Negative Moment $A_s = 2\text{-}\#6 + 1\text{-}\#4 = 1.08$ sq in.; $p = \frac{1.08}{9 \times 9.38} =$
$0.0128 < 0.0136$ (underreinforced)
from page 34, $k = 0.394$, $j = 0.869$

Max. allowable $M_s = A_s f_s jd = 1.08 \times 20{,}000 \times 0.869 \times 9.38 =$

$$176{,}000 \text{ lb-in.} = 2.08\frac{wl'^2 12}{10}$$

For $l' = 16$; $w = 276$ psf. Subtracting 54 gives 222 psf.
For $l' = 21$; $w = 160$ psf. Subtracting 54 gives 106 psf.

$$R = \frac{M}{bd^2} = \frac{176{,}000}{9 \times 9.38^2} = 222 \text{ psi, so (from page 34 for } f_s = 20{,}000 \text{ psi)}$$

$f_c < 1350$ psi and compressive reinforcement is not required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth	6″ FORMS + 2″ CONCRETE									
Joists	4″ Joists @ 24″ c/c				Wt 39 psf	5″ Joists @ 25″ c/c				Wt 42 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4
Span l′ in Ft 9	250	322	322			305				
10	201	264	286			270	333	333		
11	159	211	256	310	310	242	298	298		
12	127	171	213	263	281	200	249	269	304	304
13	103	141	176	218	235	164	205	244	277	277
14	83	116	146	183	201	136	172	205	246	246
15	67	96	122	154	174	113	143	173	209	214
16	54	79	103	131	153	95	121	147	178	189
17	43	65	86	111	135	79	102	125	153	167
18	34	54	73	95	116	65	87	107	132	150

Depth	6″ FORMS + 2½″ CONCRETE									
Joists	4″ Joists @ 24″ c/c				Wt 45 psf	5″ Joists @ 25″ c/c				Wt 48 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4
Span l′ in Ft 9	265	342				324				
10	212	280	303			286				
11	168	223	271	328	328	256	317	317		
12	133	180	227	280	297	214	264	287	322	322
13	107	147	186	232	248	174	219	260	293	293
14	86	121	155	194	212	143	182	219	260	260
15	70	100	129	163	183	119	152	184	223	226
16	55	81	107	138	160	98	128	156	190	199
17	43	67	90	118	141	82	109	132	162	176
18	34	55	76	100	122	68	91	113	139	157
19		45	63	85	105	56	77	97	121	142

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 6″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c				Wt 52 psf	5″ Joists @ 25″ c/c				Wt 55 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4
Span l′ in Ft 9	278	360								
10	223	295	319			301				
11	175	235	285	346		269				
12	139	188	238	295	313	223	279	301		
13	111	153	195	244	261	182	229	274	309	309
14	88	124	160	203	222	149	190	230	274	274
15	70	102	133	170	191	123	158	192	235	237
16	55	83	111	143	167	102	133	163	200	208
17	43	68	93	121	147	84	111	138	171	184
18	33	55	76	102	127	68	93	117	146	164
19		44	63	86	108	56	78	99	126	147
20		34	52	73	93	45	65	84	108	131

Depth: 8″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c				Wt 45 psf	5″ Joists @ 25″ c/c						Wt 48 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4
Span l′ in Ft. 10	266	345										
11	212	278	335			317						
12	171	226	282	347		266	329					
13	139	186	233	289	307	219	273	322				
14	113	154	195	243	263	183	229	274	322	322	322	322
15	93	128	164	206	229	153	193	233	281	281	281	281
16	76	107	139	175	201	129	164	199	241	248	248	248
17	62	90	118	150	179	109	140	170	208	221	221	221
18	51	75	100	129	159	92	120	147	180	199	199	199
19	41	63	85	111	138	77	102	127	157	179	179	179
20	33	52	73	96	119	65	87	110	137	163	163	163
21		43	62	83	104	55	75	95	120	144	149	149
22		35	52	72	91	46	64	83	105	127	137	137
23			44	62	80	37	55	72	92	112	126	126

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth	8" FORMS + 2½" CONCRETE											
Joists	4" Joists @ 24" c/c			Wt 51 psf		5" Joists @ 25" c/c						Wt 54 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4
Span l' in Ft												
10	277	364										
11	220	292	349			331						
12	177	237	296			280	344					
13	144	194	244	303	321	230	286	336				
14	116	160	204	254	275	191	239	287	337	337	337	337
15	95	133	171	215	238	160	201	243	292	293	293	293
16	77	111	144	183	209	134	171	207	250	259	259	259
17	62	92	122	156	185	112	145	177	216	230	230	230
18	50	77	103	133	163	94	123	151	187	207	207	207
19	40	63	88	114	142	79	105	131	162	187	187	187
20	31	52	74	99	123	66	90	113	141	169	169	169
21		43	62	84	107	55	76	98	123	149	154	154
22		34	52	72	93	45	65	84	107	131	141	141
23			43	62	80	37	55	72	93	115	130	130

Depth	8" FORMS + 3" CONCRETE											
Joists	4" Joists @ 24" c/c			Wt 58 psf		5" Joists @ 25" c/c						Wt 61 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4
Span l' in Ft												
10	288											
11	228	303				345						
12	182	245	308			291	359					
13	147	200	254	314	333	239	297	351				
14	118	164	211	264	284	198	248	298				
15	96	136	176	222	246	164	208	252	304	304	304	304
16	77	112	148	188	216	137	175	214	261	268	268	268
17	61	93	124	160	191	114	148	183	224	238	238	238
18	49	77	104	136	168	95	126	156	194	213	213	213
19	38	63	88	117	145	79	107	134	167	192	192	192
20		51	74	99	125	66	90	115	145	174	174	174
21		41	61	85	108	54	77	99	126	153	158	158
22		32	51	72	93	44	64	85	109	134	144	144
23			42	61	80	35	53	72	95	117	133	133

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 10" FORMS + 2" CONCRETE

Joists	4" Joists @ 24" c/c			Wt 50 psf		5" Joists @ 25" c/c							Wt 54 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6
Span l' in Ft 15	119	164	208	260	285	194	244	293					
16	99	138	177	222	251	164	208	251	304	308	308	308	308
17	82	117	151	191	224	139	178	217	263	275	275	275	275
18	68	99	129	165	201	118	153	187	229	247	247	247	247
19	55	84	111	143	175	100	131	162	200	225	225	225	225
20	45	70	95	124	153	86	114	141	176	204	204	204	204
21	36	60	81	108	134	72	98	123	154	185	187	187	187
22		50	70	94	118	61	85	107	136	164	172	172	172
23		41	60	82	104	52	73	94	119	145	159	159	159
24		34	51	71	91	43	62	82	105	129	147	147	147
25			43	61	80	35	53	71	93	114	137	137	137
26			36	53	70		45	62	82	102	127	127	127
27			30	46	61		38	53	72	91	112	118	118
28				39	54		31	46	63	80	101	111	111

Depth: 10" FORMS + 2½" CONCRETE

Joists	4" Joists @ 24" c/c			Wt 56 psf		5" Joists @ 25" c/c							Wt 60 psf
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6
Span l' in Ft 15	121	168	214	269	294	200	252	302					
16	100	141	181	229	259	168	213	259	313	318	318	318	318
17	82	118	154	196	230	142	182	222	270	285	285	285	285
18	67	100	132	169	206	121	157	192	235	255	255	255	255
19	54	84	112	146	179	102	134	166	205	231	231	231	231
20	44	70	96	126	156	87	115	143	179	210	210	210	210
21	34	58	82	109	137	73	99	125	157	189	192	192	192
22		48	70	95	119	61	85	109	137	167	177	177	177
23		39	59	82	104	51	73	94	121	147	163	163	163
24		31	49	70	91	41	61	82	106	130	151	151	151
25			41	61	80	34	52	71	93	115	139	139	139
26			34	52	69		44	60	81	102	126	129	129
27				44	60		36	52	71	90	113	120	120
28				37	52			44	62	80	101	112	112

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 63 psf					5″ Joists @ 25″ c/c Wt 67 psf							
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#8	#9
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#8	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6
Span l′ in Ft													
15	122	171	219	277	303	204	259	311					
16	100	143	185	235	266	172	220	266	324	328	328	328	328
17	81	119	157	201	237	144	187	228	279	292	292	292	292
18	65	99	133	173	211	122	160	196	242	262	262	262	262
19	52	83	113	148	183	103	136	169	210	237	237	237	237
20	41	68	96	128	159	86	116	146	182	215	215	215	215
21	31	56	81	110	138	72	99	127	160	192	196	196	196
22		46	68	95	120	59	85	109	139	169	180	180	180
23		37	57	81	105	49	72	94	122	149	166	166	166
24			47	69	91	39	60	81	106	131	152	152	152
25			38	59	79	30	51	69	93	116	141	141	141
26			31	50	68		41	59	81	102	127	130	130
27				42	59		34	50	70	90	113	121	121
28				34	50			42	61	79	101	112	112

Depth: 12″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 57 psf			5″ Joists @ 25″ c/c Wt 61 psf						6″ Joists @ 26″ c/c Wt 66 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Top Bar	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7
Span l′ in Ft													
18	157	200	238	227	278	294	294	294	294				
19	135	173	212	198	241	266	266	266	266	310	310	310	310
20	116	151	186	173	214	243	243	243	243	284	284	284	284
21	100	132	164	151	188	222	222	222	222	260	260	260	260
22	86	115	144	132	166	200	206	206	206	240	240	240	240
23	74	100	127	115	146	177	189	189	189	222	222	222	222
24	63	87	112	101	130	158	175	175	175	206	206	206	206
25	54	76	98	88	115	140	162	162	162	188	191	191	191
26	45	66	87	77	101	126	152	152	152	168	178	178	178
27	38	57	76	67	90	112	139	141	141	151	167	167	167
28	31	49	67	58	79	100	125	131	131	136	157	157	157
29		42	58	50	70	89	112	124	124	122	146	147	147
30		35	51	42	61	79	101	116	116	110	132	138	138
31			44	36	53	70	91	109	109	99	120	130	130
32			38	30	46	62	82	100	103	88	108	122	122
33			32		40	55	73	91	98	80	98	115	115

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 12" FORMS + 2½" CONCRETE

Joists	4" Joists @ 24" c/c Wt 63 psf			5" Joists @ 25" c/c Wt 67 psf						6" Joists @ 26" c/c Wt 72 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Top Bar	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7
Span l' in Ft													
18	159	202	244	231	283	301	301	301	301				
19	137	175	215	200	248	273	273	273	273	320	320	320	320
20	117	152	188	174	216	248	248	248	248	292	292	292	292
21	100	132	164	151	190	228	228	228	228	267	267	267	267
22	86	114	144	132	167	202	209	209	209	246	246	246	246
23	73	99	127	115	148	179	193	193	193	227	227	227	227
24	62	87	111	101	130	159	178	178	178	211	211	211	211
25	52	74	97	87	115	141	165	165	165	189	196	196	196
26	43	65	85	76	101	126	154	154	154	169	182	182	182
27	36	55	75	65	89	112	139	143	143	152	170	170	170
28		47	65	56	78	98	125	134	134	136	159	159	159
29		39	56	48	68	88	112	125	125	122	147	149	149
30		32	48	40	59	78	100	117	117	110	133	140	140
31			41	33	51	68	89	109	109	98	120	132	132
32			35		44	60	79	98	103	87	108	122	122
33					37	53	71	89	96	78	98	116	116

Depth: 12" FORMS + 3" CONCRETE

Joists	4" Joists @ 24" c/c Wt 69 psf			5" Joists @ 25" c/c Wt 74 psf						6" Joists @ 26" c/c Wt 78 psf			
Bottom Bar	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10
Truss Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9
Top Bar	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7
Span l' in Ft													
18	161	207	251	234	288	310	310	310	310				
19	137	179	219	203	251	281	281	281	281	330	330	330	330
20	117	154	191	176	219	255	255	255	255	301	301	301	301
21	101	134	167	153	192	231	233	233	233	276	276	276	276
22	86	116	146	132	168	205	214	214	214	254	254	254	254
23	72	100	128	115	148	180	197	197	197	235	235	235	235
24	61	86	112	100	130	159	182	182	182	214	217	217	217
25	51	74	98	86	114	141	168	168	168	192	202	202	202
26	41	63	85	74	99	125	154	156	156	171	187	187	187
27	32	54	74	63	87	111	138	145	145	153	175	175	175
28		45	64	54	76	98	123	136	136	137	163	163	163
29		37	55	45	66	86	110	126	126	122	147	153	153
30		31	46	37	57	75	98	118	118	109	133	143	143
31			39	30	49	66	87	109	110	97	119	134	134
32			33		41	57	77	97	103	87	107	127	127
33					34	49	68	87	97	76	96	116	119

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 14″ FORMS + 2″ CONCRETE

Joists	5″ Joists @ 25″ c/c						Wt 68 psf	6″ Joists @ 26″ c/c				Wt 73 psf
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft 21	145	179	222	260	260	260	260	306	306	306	306	306
22	126	**156**	196	236	240	240	240	283	283	283	283	283
23	110	**137**	173	210	223	223	223	262	262	262	262	262
24	95	**120**	154	188	207	207	207	243	243	243	243	243
25	82	**106**	137	166	192	192	192	223	227	227	227	227
26	71	**92**	**121**	149	179	179	179	201	212	212	212	212
27	61	**81**	**107**	134	166	167	167	180	**198**	**198**	**198**	**198**
28	52	**70**	**95**	120	150	156	156	163	**186**	**186**	**186**	**186**
29	44	**61**	**84**	107	134	147	147	147	**174**	**174**	**174**	**174**
30	36	**53**	**74**	**96**	121	137	137	133	**160**	**164**	**164**	**164**
31	30	**45**	**65**	**85**	109	**129**	**129**	119	**146**	**155**	**155**	**155**
32		**38**	**59**	**76**	98	**120**	**122**	107	**132**	**146**	**146**	**146**
33		**32**	**50**	**67**	89	**109**	**115**	**97**	**120**	**138**	**138**	**138**
34			**43**	**59**	79	**99**	**108**	**87**	**109**	**128**	**130**	**130**
35			**37**	**52**	**71**	**89**	**102**	**78**	**98**	**117**	**123**	**123**
36			**31**	**46**	**64**	**81**	**97**	**70**	**89**	**107**	**117**	**117**
37				**40**	**61**	**73**	**91**	**62**	**80**	**97**	**111**	**111**
38				**34**	**50**	**65**	**83**	**55**	**72**	**88**	**105**	**105**

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 25″ c/c						Wt 75 psf	6″ Joists @ 26″ c/c				Wt 80 psf
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft 21	142	178	223	265	265	265	265	312	312	312	312	312
22	123	156	197	237	244	244	244	288	288	288	288	288
23	106	136	174	211	225	225	225	267	267	267	267	267
24	91	119	154	187	208	208	208	247	247	247	247	247
25	79	104	136	167	193	193	193	223	230	230	230	230
26	67	90	**120**	148	179	179	179	200	215	215	215	215
27	56	78	**106**	132	164	167	167	180	200	200	200	200
28	47	68	**93**	118	147	156	156	162	188	**188**	**188**	**188**
29	39	58	**82**	105	131	147	147	145	175	**176**	**176**	**176**
30	32	50	**71**	**93**	119	138	138	131	158	**165**	**165**	**165**
31		41	**62**	**82**	106	129	129	117	143	**155**	**155**	**155**
32		34	**54**	**72**	95	118	**121**	105	129	**147**	**147**	**147**
33			**46**	**64**	85	106	**114**	**94**	117	**138**	**138**	**138**
34			**39**	**56**	75	95	**107**	**84**	106	**126**	**130**	**130**
35			**33**	**48**	**67**	86	**101**	**75**	95	**115**	**123**	**123**
36				**42**	**60**	77	**95**	**66**	86	**104**	**116**	**116**
37				**36**	**52**	69	**88**	**59**	**77**	**94**	**110**	**110**
38				**30**	**45**	61	**79**	**51**	**68**	**85**	**104**	**104**

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth	14″ FORMS + 3″ CONCRETE											
Joists	5″ Joists @ 25″ c/c Wt 81 psf							6″ Joists @ 26″ c/c Wt 86 psf				
Bottom Bar	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft 21	143	180	225	269	269	269	269	318	318	318	318	318
22	123	157	198	240	247	247	247	293	293	293	293	293
23	105	136	175	212	228	228	228	271	271	271	271	271
24	91	119	153	188	211	211	211	251	251	251	251	251
25	77	103	135	168	196	196	196	225	233	233	233	233
26	65	89	119	148	182	182	182	201	217	217	217	217
27	55	77	105	132	164	169	169	181	203	203	203	203
28	45	66	91	117	147	158	158	162	190	190	190	190
29	37	56	80	103	131	148	148	145	175	178	178	178
30		47	69	91	118	138	138	130	157	167	167	167
31		39	60	85	105	129	129	116	142	156	156	156
32		31	51	70	93	116	122	104	128	148	148	148
33			44	62	84	105	114	93	116	138	139	139
34			36	53	74	94	107	82	104	126	131	131
35				46	65	84	100	73	94	113	123	123
36				39	57	75	94	64	84	103	117	117
37				33	50	67	86	56	75	93	110	110
38					43	59	77	49	66	83	104	104

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 139-140.

Depth	6″ FORMS + 2½″ CONCRETE											
Joists	4″ Joists @ 34″ c/c Wt 41 psf					5″ Joists @ 35″ c/c Wt 43 psf					6″ Joists @ 36″ c/c Wt 45 psf	
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#6	#7
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#5	#4
Span l′ in Ft 9	177	232	232	281	281	223	277	277	309	309	316	316
10	140	189	205	248	248	195	244	244	274	274	280	280
11	109	148	182	222	222	174	218	218	245	245	251	251
12	85	118	151	189	201	144	180	197	222	222	226	226
13	66	95	122	155	166	116	148	177	201	201	205	205
14	52	76	100	128	141	93	121	148	177	177	171	187
15	39	61	82	106	120	76	100	123	151	153	143	172
16	30	48	66	88	104	61	83	103	127	134	120	143
17		38	54	74	90	50	69	86	107	117	101	122
18			39	61	77	40	56	72	91	103	85	104
19			35	51	65	31	46	60	78	93	72	93

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 6″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c, Wt 48 psf					5″ Joists @ 35″ c/c, Wt 50 psf					6″ Joists @ 36″ c/c, Wt 52 psf	
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#7	#7
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#6	#7
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#5	#4
Span l′ in Ft 9	184	243	243	295	295	233	289	289	325	325	334	334
10	146	197	214	261	261	204	256	256	288	288	295	295
11	112	154	190	233	233	181	228	228	257	257	263	263
12	87	121	157	197	210	149	188	204	232	232	236	236
13	67	97	126	161	173	119	153	185	210	210	214	214
14	51	76	102	132	145	96	125	154	185	185	179	196
15	38	61	82	102	123	77	102	126	157	158	149	177
16		47	67	89	107	62	84	106	132	138	125	150
17		36	54	74	92	49	68	88	111	121	105	127
18			42	61	78	38	56	73	93	106	87	107
19			33	49	65		45	60	79	94	73	91
20				40	54		36	49	66	83	61	77

Depth: 8″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c, Wt 45 psf					5″ Joists @ 35″ c/c, Wt 48 psf						6″ Joists @ 36″, Wt 50 psf		
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#7	#7	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4
Span l′ in Ft 10	187	248	266	322	322	254	314	314						
11	146	197	237	289	289	227	282	282	316	316	316	324		
12	116	158	200	248	261	190	236	254	286	286	286	292	310	310
13	92	128	163	205	217	155	195	231	260	260	260	267	283	283
14	73	104	135	170	185	127	161	195	231	231	231	244	260	260
15	58	85	112	143	159	105	134	164	199	200	200	224	239	239
16	45	69	92	120	138	86	113	138	169	175	175	192	210	210
17	35	56	77	101	121	70	94	117	145	155	155	164	187	187
18		45	64	85	106	57	78	98	124	138	138	142	166	166
19		35	56	71	91	50	66	84	106	124	124	122	147	150
20			43	61	78	38	55	71	90	111	111	105	128	136
21			35	50	66	30	45	60	78	97	100	91	111	123
22				42	57		37	50	67	84	91	78	97	113
23				35	47		30	42	57	72	83	67	84	100

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 8″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 51 psf					5″ Joists @ 35″ c/c Wt 54 psf						6″ Joists @ 36″ c/c Wt 56 psf		
Bottom Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#8	#7	#8	#8
Truss Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#7	#7	#7	#8
Top Bar	#4	#4	#4	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4
Span l′ in Ft 10	181	258	276			265	328	328						
11	142	204	247	301	301	236	293	293	330	330	330			
12	111	163	207	258	271	196	246	264	298	298	298	305	324	324
13	87	131	169	212	225	160	202	244	272	272	272	277	295	295
14	68	106	139	176	200	131	167	202	240	240	240	254	270	270
15	53	86	114	146	163	107	138	170	207	207	207	233	248	248
16	40	69	94	123	142	87	115	142	176	181	181	200	218	218
17		55	77	103	125	71	95	120	150	160	160	171	193	193
18		47	63	86	109	57	79	101	128	142	142	146	173	173
19		37	52	73	92	46	66	85	109	127	127	125	152	155
20			42	60	78	36	54	72	93	114	114	108	126	140
21			33	50	66		44	60	79	99	102	92	114	127
22				41	55		35	50	67	85	92	79	99	115
23				33	46			41	57	73	84	68	86	104

Depth: 10″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 50 psf				5″ Joists @ 35″ c/c Wt 52 psf						6″ Joists @ 36″ c/c Wt 55 psf				
Bottom Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#4	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft 15	108	140	179	197	207	249	249	249	249	249	295	295	295	295	295
16	89	117	151	172	176	214	218	218	218	218	260	260	260	260	260
17	73	98	128	152	149	184	194	194	194	194	232	232	232	232	232
18	60	83	109	135	128	159	173	173	173	173	207	207	207	207	207
19	49	69	92	116	110	137	156	156	156	156	187	187	187	187	187
20	39	57	78	100	93	119	141	141	141	141	170	170	170	170	170
21	30	47	66	86	80	103	126	128	128	128	155	155	155	155	155
22		39	57	73	69	89	110	117	117	117	142	142	142	142	142
23		31	47	63	58	77	96	107	107	107	130	130	130	130	130
24			39	54	49	67	84	99	99	99	117	120	120	120	120
25			32	46	42	57	73	90	90	90	103	110	110	110	110
26				38	34	49	64	83	83	83	91	102	102	102	102
27				32		42	55	72	77	77	81	95	95	95	95
28						35	48	63	71	71	71	87	88	88	88

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 56 psf				5″ Joists @ 35″ c/c Wt 58 psf						6″ Joists @ 36″ c/c Wt 61 psf				
Bottom Bar	#5	#5	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#4	#5	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft															
15	109	143	184	202	212	255	255	255	255	255	304	304	304	304	304
16	89	119	154	176	180	221	224	224	224	224	268	268	268	268	268
17	72	99	130	156	153	189	198	198	198	198	238	238	238	238	238
18	58	82	111	137	130	162	177	177	177	177	213	213	213	213	213
19	47	68	93	118	111	140	159	159	159	159	192	192	192	192	192
20	37	56	79	101	94	120	144	144	144	144	174	174	174	174	174
21		46	66	86	81	104	127	130	130	130	158	158	158	158	158
22		37	55	73	68	89	110	118	118	118	144	144	144	144	144
23		32	46	63	57	77	96	108	108	108	132	132	132	132	132
24			37	53	45	65	83	98	98	98	118	122	122	122	122
25			30	44	39	56	73	90	90	90	104	112	112	112	112
26				37	32	48	63	81	83	83	91	103	103	103	103
27				30		40	54	70	76	76	80	95	95	95	95
28						33	46	63	70	70	70	87	88	88	88

Depth: 12″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 54 psf		5″ Joists @ 35″ c/c Wt 57 psf						6″ Joists @ 36″ c/c Wt 61 psf				
Bottom Bar	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft													
18	133	163	156	193	207	207	207	207	247	247	247	247	247
19	114	142	134	168	186	186	186	186	223	223	223	223	223
20	98	123	115	145	169	169	169	169	202	202	202	202	202
21	83	106	99	126	154	154	154	154	185	185	185	185	185
22	71	92	85	110	135	141	141	141	169	169	169	169	169
23	60	80	73	96	119	130	130	130	156	156	156	156	156
24	52	69	63	84	104	119	119	119	143	144	144	144	144
25	43	59	53	73	91	110	110	110	127	133	133	133	133
26	36	50	45	63	81	102	101	101	113	124	124	124	124
27		43	37	54	71	90	93	93	101	115	115	115	115
28		36	31	46	61	80	87	87	89	107	107	107	107
29		30		39	53	71	81	81	79	97	100	100	100
30				33	46	62	75	75	70	87	93	93	93
31					39	54	69	70	62	77	87	87	87
32					34	47	61	65	54	69	81	81	81
33						41	54	61	47	62	75	76	76

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 155-157.

Depth: 12″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 60 psf		5″ Joists @ 35″ c/c Wt 63 psf						6″ Joists @ 36″ c/c Wt 67 psf				
Bottom Bar	#6	#6	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#10
Truss Bar	#5	#6	#6	#6	#7	#7	#8	#8	#8	#8	#9	#9	#10
Top Bar	#4	#4	#4	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5
Span l′ in Ft													
18	135	166	157	196	211	211	211	211	251	251	251	251	251
19	115	143	135	169	188	188	188	188	227	227	227	227	227
20	97	123	115	146	172	172	172	172	204	204	204	204	204
21	83	107	99	127	155	155	155	155	187	187	187	187	187
22	70	92	84	110	136	142	142	142	172	172	172	172	172
23	59	79	72	95	118	130	130	130	158	158	158	158	158
24	49	68	61	83	103	119	119	119	144	145	145	145	145
25	41	58	51	71	90	110	110	110	128	134	134	134	134
26	33	49	42	60	79	100	101	101	113	124	124	124	124
27		41	35	52	69	88	93	93	100	114	114	114	114
28		34		45	60	78	86	86	88	106	106	106	106
29				37	51	68	79	79	77	96	99	99	99
30				31	43	60	73	73	68	85	92	92	92
31					37	52	67	68	59	75	86	86	86
32					31	45	59	62	52	66	79	79	79
33						38	52	58	44	59	73	74	74

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 35″ c/c Wt 62 psf						6″ Joists @ 36″ c/c Wt 66 psf				7″ Joists @ 37″ c/c Wt 70 psf				
Bottom Bar	#7	#7	#8	#8	#9	#9	#9	#9	#10	#10	#9	#10	#10	#11	#11
Truss Bar	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#9	#9	#10	#10	#11
Top Bar	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5	#4	#7	#5	#7	#5
Span l′ in Ft															
21	151	181	181	181	181	181	217	217	217	217	251	251	251	251	251
22	132	161	166	166	166	166	199	199	199	199	231	231	231	231	231
23	116	142	152	152	152	152	184	184	184	184	213	213	213	213	213
24	101	125	140	140	140	140	170	170	170	170	197	197	197	197	197
25	89	111	129	129	129	129	158	158	158	158	183	183	183	183	183
26	77	97	119	119	119	119	147	147	147	147	171	171	171	171	171
27	67	86	109	111	111	111	137	137	137	137	159	159	159	159	159
28	58	76	96	103	103	103	128	128	128	128	144	149	149	149	149
29	50	66	85	97	97	97	118	119	119	119	129	139	139	139	139
30	42	58	77	90	90	90	106	111	111	111	116	131	131	131	131
31	36	50	67	84	84	84	95	104	104	104	104	122	122	122	122
32	30	43	59	76	78	78	85	98	98	98	93	115	116	116	116
33		37	52	67	73	73	76	92	92	92	84	104	108	108	108
34		32	45	59	68	68	68	83	86	86	75	93	102	102	102
35			39	53	64	64	60	74	81	81	67	84	96	96	96
36			34	46	59	59	54	67	76	76	59	76	90	90	90
37				41	54	55	48	59	71	71	52	68	84	85	85
38				35	48	52	41	53	67	67	46	61	76	80	80

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
END SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

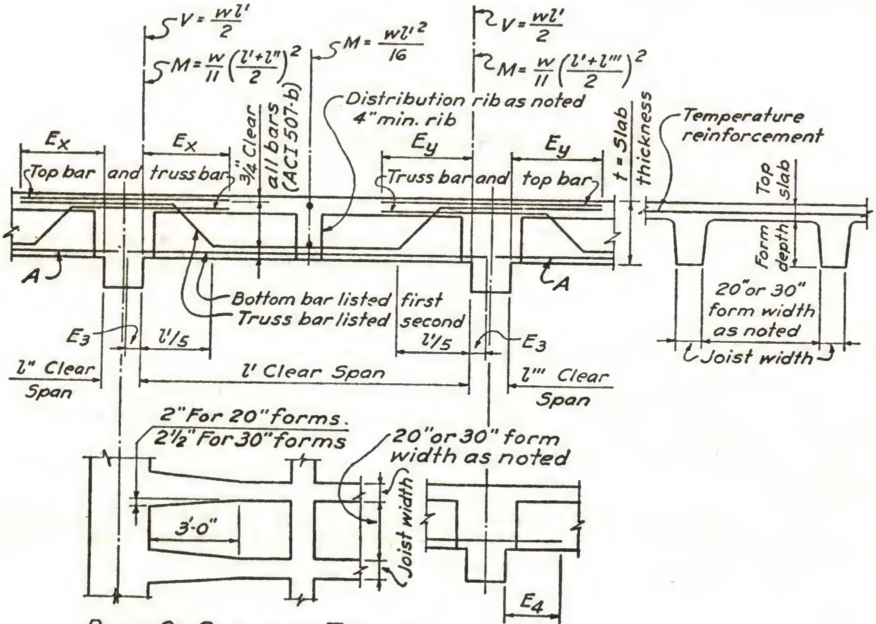
For limitations and explanation of use of tables, see pages 155-157.

Depth	14″ FORMS + 3″ CONCRETE														
Joists	5″ Joists @ 35″ c/c Wt 68 psf						6″ Joists @ 36″ c/c Wt 72 psf				7″ Joists @ 37″ c/c Wt 76 psf				
Bottom Bar	#7	#7	#8	#8	#9	#9	#9	#9	#10	#10	#9	#10	#10	#11	#11
Truss Bar	#6	#7	#7	#8	#8	#9	#8	#9	#9	#10	#9	#9	#10	#10	#11
Top Bar	#5	#4	#6	#4	#6	#4	#6	#4	#7	#5	#4	#7	#5	#7	#5
Span l′ in Ft 21	150	182	182	182	182	182	220	220	220	220	256	256	256	256	256
22	131	161	166	166	166	166	201	201	201	201	236	236	236	236	236
23	115	141	153	153	153	153	186	186	186	186	217	217	217	217	217
24	99	124	141	141	141	141	171	171	171	171	200	200	200	200	200
25	86	110	130	130	130	130	158	158	158	158	186	186	186	186	186
26	75	96	119	119	119	119	147	147	147	147	173	173	173	173	173
27	65	84	107	111	111	111	137	137	137	137	160	161	161	161	161
28	55	74	95	103	103	103	127	127	127	127	144	150	150	150	150
29	47	64	84	95	95	95	116	119	119	119	128	141	141	141	141
30	39	55	74	88	88	88	104	110	110	110	115	131	131	131	131
31	33	51	65	82	82	82	93	103	103	103	102	123	123	123	123
32		40	56	73	77	77	82	97	97	97	91	113	115	115	115
33		34	50	65	71	71	74	90	90	90	81	102	108	108	108
34			42	57	66	66	65	81	84	84	73	91	102	102	102
35			36	50	61	61	57	72	79	79	64	82	96	96	96
36			30	44	57	57	51	64	74	74	57	74	90	90	90
37				38	51	53	44	57	69	69	50	66	82	85	85
38				32	45	49	38	50	65	65	43	58	74	80	80

Tabulated values in boldface type require embedment E_4 explained on page 155.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION—INTERIOR SPAN

Read the general explanation of the arrangement of tables for Concrete Joist Construction on pages 136 to 138 before using these tables for interior spans. The details of temperature reinforcement, tapered end forms, distribution ribs, and especially the type of deformed bars all apply equally well here.



PLAN OF STANDARD TAPERED END WHERE REQUIRED

STRESSES:—	
f'_c =	3000 psi
f_c =	1350 psi
f_s =	20,000 psi
v_c =	90 psi *
u =	300 psi *

CODES:—"Building Code Requirements for Reinforced Concrete (ACI 318-56)"; "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

E_3 = bottom bar to extend 6 in. into the support except when values in the load tables are printed in boldface type.

E_4 = When the values in the load tables are printed in boldface type, bottom bar should extend not less than 17 bar diameters nor less than $l'''/10$ past the far face of the support. Embedment of bottom bar at interior support is determined by the fact that the bottom bar is required for compressive reinforcement. The exact length varies. The maximum is that which will develop the full compression in the bar at the higher unit stress permitted by the ACI Code (20,000 psi) and which will at the same time extend the needed distance across the moment curve. The capacity

* Bond and diagonal tension values are based upon deformed bars meeting ASTM A305. Plain round bars or deformed bars not meeting ASTM A305 will not give sufficient bond resistance.

CONCRETE JOIST CONSTRUCTION—INTERIOR SPAN

of the joist may be determined by shear, bond or flexure. The recommendation for E_4 will cover the worst condition. The user may at his option work out the needs of any particular problem (see page 173).

$$E_x = \text{not less than} \begin{cases} l'/4 \\ l''/4 \\ \text{17 bar diameters} \\ \text{past bend-down} \\ \text{point (24 dia.} \\ \text{when } d > 12 \text{ in.).} \end{cases} \text{whichever is greatest.}$$

(ACI 902(a) requires top bars to extend to $l'/16$, d, or half bond length past point of inflection.)

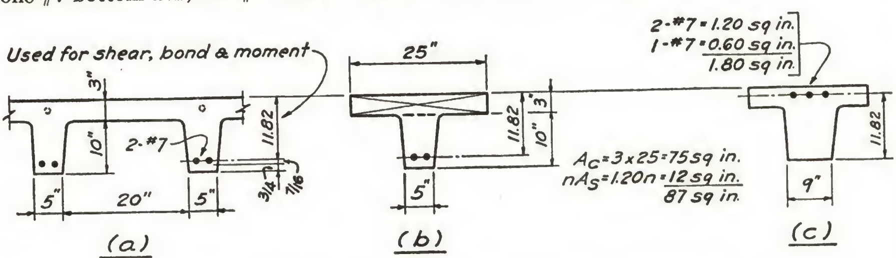
$$E_y = \text{not less than} \begin{cases} l'/4 \\ l'''/4 \\ \text{17 bar diameters} \\ \text{past bend-down} \\ \text{point (24 dia.} \\ \text{when } d > 12 \text{ in.).} \end{cases} \text{whichever is greatest.}$$

The top bar in the table is scheduled on the basis of the adjoining span providing a bent bar of area equal to that of the bent bar in the span under consideration; any considerable variation in negative moment by reason of changes in load, span length, or end restraint of the adjacent span must be worked out by the general principles of continuity (pages 66-81).

A = bottom bar in adjoining span, not shown.

Almost all usual combinations of form depth, top slab and reinforcement are presented herewith. To show how the tables for interior spans were computed and to permit extension of the tables if required, an illustrative example is shown:—

Example—Determine the safe carrying capacity on spans of 18 and 26 feet of 10 in. deep forms plus 3 in. top slab with 5 in. wide joists at 25 in. centers and reinforced with one #7 bottom bar, one #7 truss bar and one #7 top bar. (See page 178.)



Shear Max. allowable $V = v_c bjd = 90 \times 5 \times 7/8 \times 11.82 = 4655$ lb.

$$2.08 \times \frac{wl'}{2} = 4655 \text{ lb}$$

For $l' = 18$; $w = 249$ psf. Subtracting 67 gives 182 psf.
For $l' = 26$; $w = 172$ psf. Subtracting 67 gives 105 psf.

Bond Max. allowable $V = \Sigma ojdu = 2.749 \times 7/8 \times 11.82 \times 300 =$

$$8530 \text{ lb} = 2.08 \times \frac{wl' \times 0.70\,*}{2}$$

For $l' = 18$; $w = 650$ psf. Subtracting 67 gives 583 psf.
For $l' = 26$; $w = 450$ psf. Subtracting 67 gives 383 psf.

Positive Moment A_s = 2-#7 = 1.20 sq in.
Since $t = 3$ in. it is quite likely that the neutral axis lies within the flange, if so:

* See note on bottom of page 173.

CONCRETE JOIST CONSTRUCTION—INTERIOR SPAN

$p = \dfrac{1.20}{25 \times 11.82} = 0.00406 < 0.0136$ (underreinforced)

from page 34, $k = 0.247$; $kd = 2.92$ in. < 3 in.; $j = 0.917$

Max. allowable $M_s = A_s f_s jd = 1.20 \times 20{,}000 \times 0.917 \times 11.82 =$

$$260{,}000 \text{ lb-in.} = 2.08 \times \frac{wl'^2 \times 12}{16}$$

For $l' = 18$; $w = 513$ psf. Subtracting 67 gives 446 psf.
For $l' = 26$; $w = 246$ psf. Subtracting 67 gives 179 psf.

Tapered End Shear

Max. allowable $V = v_c bjd = 90 \times 9 \times \frac{7}{8} \times 11.82 = 8360$ lb $=$

$$2.08 \times \frac{wl'}{2}$$

For $l' = 18$; $w = 447$ psf. Subtracting 67 gives 380 psf.
For $l' = 26$; $w = 309$ psf. Subtracting 67 gives 242 psf.

Shear at Root of Tapered End

$V = 5 \times 11.82 \times \frac{7}{8} \times 90 = 4655 = 2.08 \times \dfrac{w(l' - 6.15)}{2}$

For $l' = 18$; $w = 378$ psf. Subtracting 67 gives 311 psf
For $l' = 26$; $w = 226$ psf. Subtracting 67 gives 159 psf } *In table on page 178*

Negative Moment

$A_s = 3\text{-}\#7 = 1.80$ sq in.; $p = \dfrac{1.80}{9 \times 11.82} = 0.0169 > 0.0136$ ($f_c > 1350$)

From page 34, $j = 0.854$

Max. allowable $M = A_s f_s jd = 1.80 \times 20{,}000 \times 0.854 \times 11.82 =$

$$363{,}500 \text{ lb-in.} = 2.08 \times \frac{wl'^2 \times 12}{11}$$

For $l' = 18$; $w = 494$ psf. Subtracting 67 gives 427 psf.
For $l' = 26$; $w = 237$ psf. Subtracting 67 gives 170 psf.

Tapered End Compression

For $l' = 18$; $w = 378$; $-M = \dfrac{2.08 \times 378 \times 18 \times 18 \times 12}{11} = 278{,}000$ lb-in.

For $l' = 26$; $w = 226$; $-M = \dfrac{2.08 \times 226 \times 26 \times 26 \times 12}{11} = 347{,}000$ lb-in.

$R = \dfrac{M}{bd^2} = \dfrac{347{,}000}{9 \times 11.82 \times 11.82} = 276 > 236$, so, from page 34 when $f_s = 20{,}000$, $f_c > 1350$ and bottom bars must be extended to act as compressive reinforcement.

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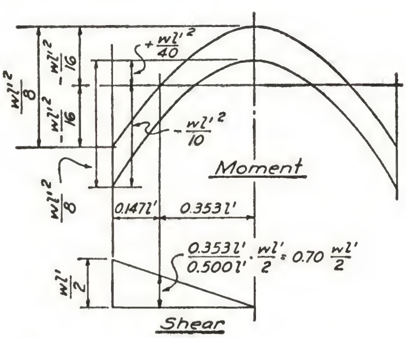
* 0.70 is a factor to represent the shear at the point of inflection where the bond on the bottom bar is a maximum, see figure below.



CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 8″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 51 psf				5″ Joists @ 25″ c/c Wt 54 psf			6″ Joists @ 26″ c/c Wt 57 psf		
Bottom Bar	#4	#4	#5	#5	#5	#5	#6	#5	#5	#6
Truss Bar	#4	#5	#5	#6	#5	#6	#6	#5	#6	#6
Top Bar	#4	#5	#5	#5	#5	#5	#6	#5	#5	#6
Span l′ in Ft										
11	343									
12	280									
13	231	306						340		
14	192	256	320		302			285	352	
15	161	217	272	335	256	318		241	299	
16	135	184	233	288	219	272	326	205	256	308
17	114	157	200	249	187	234	282	175	220	267
18	96	135	173	217	161	203	246	150	190	231
19	81	116	150	189	139	176	215	129	165	201
20	68	99	131	166	121	154	189	111	143	177
21	57	85	113	146	103	135	166	94	125	154
22	47	73	99	128	90	118	147	81	108	136
23	39	63	86	113	78	103	129	69	94	119

Depth: 8″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 58 psf				5″ Joists @ 25″ c/c Wt 61 psf			6″ Joists @ 26″ c/c Wt 64 psf		
Bottom Bar	#4	#4	#5	#5	#5	#5	#6	#5	#5	#6
Truss Bar	#4	#5	#5	#6	#5	#6	#6	#5	#6	#6
Top Bar	#4	#5	#5	#5	#5	#5	#6	#5	#5	#6
Span l′ in Ft										
11	358									
12	291									
13	240	318						354		
14	199	266	333		314			297		
15	166	224	283		266	329		251	310	
16	138	190	241	300	226	283	339	212	266	324
17	116	162	207	258	193	242	293	181	228	277
18	97	138	178	224	166	210	256	154	196	241
19	81	118	154	195	143	182	223	132	170	209
20	68	101	133	171	122	159	195	112	147	182
21	56	86	115	149	105	138	171	96	127	159
22	46	73	100	131	91	120	151	82	110	140
23	37	62	87	115	78	105	133	70	96	122

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth	10″ FORMS + 2″ CONCRETE											
Joists	4″ Joists @ 24″ c/c Wt 50 psf					5″ Joists @ 25″ c/c Wt 54 psf				6″ Joists @ 26″ c/c Wt 58 psf		
Bottom Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6
Truss Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#6	#6	#7
Top Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6
Span l′ in Ft												
15	197	259	328	334	334	309						
16	167	222	282	296	296	264	326			308		
17	142	191	244	264	264	228	283	324	324	266	320	
18	121	165	212	238	238	196	246	292	292	231	279	337
19	104	143	185	216	216	172	216	260	264	201	244	296
20	89	124	162	197	197	149	189	230	243	176	215	262
21	76	109	143	180	180	131	167	203	223	154	189	232
22	65	95	126	160	166	115	148	181	206	136	168	207
23	55	83	109	142	153	99	130	161	190	119	149	185
24	46	72	96	126	142	87	115	143	177	104	131	164
25	39	62	85	112	132	76	102	127	159	92	116	147
26	32	54	75	100	123	66	90	114	143	80	104	131
27		46	65	89	112	57	79	102	128	70	92	117
28		39	57	79	101	49	70	91	116	62	81	105

Depth	10″ FORMS + 2½″ CONCRETE											
Joists	4″ Joists @ 24″ c/c Wt 56 psf					5″ Joists @ 25″ c/c Wt 60 psf				6″ Joists @ 26″ c/c Wt 64 psf		
Bottom Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6
Truss Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#6	#6	#7
Top Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6
Span l′ in Ft												
15	202	270	337									
16	171	231	289	306	306	271	338			319		
17	145	198	250	273	273	234	292			275	330	
18	123	171	217	246	246	202	255	302	302	239	289	
19	105	147	189	222	222	175	222	268	276	207	252	306
20	90	128	165	202	202	152	194	237	250	181	221	270
21	76	111	144	185	185	132	171	209	230	158	194	239
22	64	96	126	163	170	115	150	186	212	138	172	212
23	54	83	112	145	157	101	133	165	196	122	152	188
24	45	72	98	128	145	87	117	145	181	106	134	168
25	37	62	86	114	134	76	103	130	162	93	119	149
26	30	52	75	101	125	66	91	116	145	81	105	134
27		45	66	90	114	57	80	103	131	73	92	120
28		38	57	80	102	48	70	91	117	61	81	106

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 63 psf					5″ Joists @ 25″ c/c Wt 67 psf					6″ Joists @ 26″ c/c Wt 71 psf			
Bottom Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#6	#6	#7
Truss Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#6	#6	#7	#7
Top Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#6	#6	#7
Span l′ in Ft														
15	206	277	348			328								
16	173	236	298	315	315	280					330			
17	146	202	257	281	281	240	303				284	342		
18	124	173	222	253	253	207	262	311	311	311	246	296		
19	105	149	193	229	229	179	229	277	283	283	213	259	315	
20	89	128	168	207	207	155	200	243	257	257	186	227	278	304
21	75	111	146	189	189	134	175	214	235	235	162	199	246	281
22	63	96	128	167	174	116	154	189	216	216	141	175	217	257
23	52	82	112	147	160	101	135	167	199	199	123	154	193	231
24	43	70	98	130	147	87	119	148	185	185	107	136	171	206
25	34	60	85	115	136	75	104	131	165	172	93	119	152	183
26		51	74	102	126	65	91	116	147	159	81	105	135	164
27		42	64	90	114	55	79	103	132	149	70	92	120	148
28		35	55	79	102	46	69	91	118	139	60	81	107	132

Depth: 12″ FORMS + 2″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 57 psf				5″ Joists @ 25″ c/c Wt 61 psf					6″ Joists @ 26″ c/c Wt 66 psf					
Bottom Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#7
Truss Bar	#5	#5	#6	#6	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#8
Top Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#6
Span l′ in Ft															
18	200	253	283	283	237	297				220	278	335			
19	173	221	258	258	206	261	314	317	317	191	243	294			
20	151	194	235	235	180	229	277	291	291	166	213	259	317		
21	132	171	215	215	158	202	246	267	267	144	187	228	281	311	311
22	115	151	193	199	139	179	218	247	247	126	165	202	251	288	288
23	100	133	171	183	121	158	194	229	229	109	144	180	224	268	268
24	87	117	152	170	106	140	173	213	213	95	127	159	200	240	248
25	76	104	136	158	94	124	155	194	198	83	112	142	180	216	232
26	66	92	121	147	82	110	139	174	186	71	98	126	160	195	217
27	57	81	109	136	71	98	124	158	174	61	87	112	144	177	204
28	49	72	98	122	62	87	111	143	163	53	77	99	130	159	192
29	42	63	87	111	54	77	100	128	153	44	67	89	116	144	176
30	36	55	78	100	46	68	89	116	143	37	58	78	104	131	160
31	30	48	69	90	39	60	80	104	130	30	50	69	93	118	145
32		41	61	81	33	52	71	95	118		43	61	84	106	133
33		36	55	73		46	64	85	107		37	54	75	96	121

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 12″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 63 psf				5″ Joists @ 25″ c/c Wt 67 psf					6″ Joists @ 26″ c/c Wt 72 psf					
Bottom Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#7
Truss Bar	#5	#5	#6	#6	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#8
Top Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#6
Span l′ in Ft															
18	204	259	291	291	242	304				225	285				
19	176	226	264	264	210	266	322	326	326	195	248	302			
20	153	198	240	240	**184**	233	283	299	299	169	**217**	265	325		
21	**133**	174	220	220	**161**	206	251	274	274	147	**190**	234	288	320	320
22	**115**	153	196	203	**140**	182	223	252	253	127	**167**	207	256	**296**	**296**
23	**100**	134	174	187	**122**	161	198	234	234	110	**147**	**183**	228	**272**	**274**
24	**87**	118	154	173	**107**	**141**	177	217	217	95	**128**	**162**	204	**244**	**255**
25	**75**	**104**	137	161	**93**	**125**	158	197	**202**	82	**113**	**144**	182	**219**	**237**
26	**65**	**92**	122	150	**81**	**111**	140	177	**188**	71	**99**	**127**	162	**197**	**222**
27	**56**	**81**	109	137	**71**	**98**	125	160	**177**	60	**87**	**113**	**146**	**177**	**208**
28	**47**	**70**	97	123	**61**	**86**	**112**	143	**165**	51	**75**	**100**	**130**	**160**	**195**
29	**40**	**61**	86	111	**52**	**76**	**100**	129	**155**	42	**66**	**89**	**116**	**144**	**177**
30	**33**	**53**	**76**	100	**44**	**66**	**89**	116	**143**	35	**56**	**78**	**104**	**130**	**162**
31		**46**	**68**	90	**37**	**58**	**79**	105	**130**		**48**	**69**	**93**	**117**	**146**
32		**39**	**60**	80	**31**	**51**	**70**	94	**118**		**41**	**60**	**83**	**106**	**133**
33		**33**	**52**	72		**43**	**62**	**85**	**107**		**34**	**52**	**74**	**95**	**121**

Depth: 12″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 24″ c/c Wt 69 psf				5″ Joists @ 25″ c/c Wt 74 psf					6″ Joists @ 26″ c/c Wt 78 psf					
Bottom Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#7
Truss Bar	#5	#5	#6	#6	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#8
Top Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#6
Span l′ in Ft															
18	207	265	299	299	247	311				230	292				
19	179	231	270	270	214	272	329			199	254	310			
20	155	202	246	246	**186**	238	290	304	304	172	**222**	272	333		
21	**134**	176	225	225	**161**	209	255	279	279	148	**194**	239	294		
22	**116**	155	200	207	**141**	184	226	256	256	129	**170**	211	262	303	303
23	**100**	136	177	191	**123**	162	202	237	237	111	**149**	**187**	233	277	**280**
24	**87**	119	157	176	**106**	**143**	179	220	220	96	**131**	**165**	207	248	**261**
25	**74**	**105**	139	163	**93**	**126**	158	200	205	89	**114**	**145**	185	223	**243**
26	**64**	**91**	124	151	**80**	**111**	141	178	190	70	**100**	**129**	165	200	**227**
27	**54**	**80**	110	139	**69**	**97**	126	160	**178**	59	**87**	**114**	**147**	180	**212**
28	**45**	**69**	97	125	**58**	**85**	**112**	144	**166**	49	**75**	**101**	**132**	162	**198**
29	**38**	**60**	86	112	**49**	**75**	**99**	129	**156**	41	**65**	**89**	**117**	145	**180**
30	**31**	**52**	**76**	100	**42**	**65**	**88**	116	**143**	33	**55**	**78**	**105**	131	**163**
31		**44**	**67**	89	**34**	**56**	**78**	104	**130**		**47**	**68**	**93**	**118**	**147**
32		**37**	**58**	79		**48**	**68**	93	**118**		**39**	**59**	**83**	**106**	**133**
33		**31**	**51**	71		**41**	**60**	**83**	**106**		**32**	**51**	**73**	**95**	**121**

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 14″ FORMS + 2″ CONCRETE

Joists	5″ Joists @ 25″ c/c Wt 68 psf					6″ Joists @ 26″ c/c Wt 73 psf					7″ Joists @ 27″ c/c Wt 78 psf				
Bottom Bar	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#6	#6	#7	#7	#8
Truss Bar	#6	#6	#7	#7	#8	#6	#6	#7	#7	#8	#6	#7	#7	#8	#8
Top Bar	#5	#6	#6	#7	#6	#5	#6	#6	#7	#6	#6	#6	#7	#6	#8
Span l′ in Ft 21	238	290	310	310	310	221	271	333			254	313			
22	211	257	286	286	286	196	240	297			223	278	331		
23	187	230	265	265	265	173	213	266	312	312	198	248	296		
24	167	206	247	247	247	153	**190**	237	284	291	175	**221**	266	318	
25	**148**	184	230	231	231	135	**170**	213	257	**269**	156	**198**	239	288	**308**
26	**132**	165	208	215	215	119	**151**	192	231	**254**	138	**177**	214	261	**289**
27	**118**	148	187	202	202	106	**135**	173	209	**239**	122	**158**	**194**	236	**272**
28	**104**	**133**	169	190	190	93	**120**	**155**	189	**224**	108	**142**	**175**	214	**253**
29	**93**	**120**	153	**178**	**178**	82	**107**	**139**	172	**209**	96	**126**	**158**	194	**230**
30	**82**	**107**	138	**168**	**168**	71	**95**	**125**	156	**191**	84	**113**	**142**	176	**210**
31	**73**	**96**	125	**155**	**158**	62	**85**	**113**	141	**174**	74	**101**	**128**	160	**191**
32	**64**	**86**	113	**141**	**150**	54	**75**	**101**	**128**	**158**	65	**90**	**116**	**144**	**175**
33	**56**	**77**	**103**	**129**	**142**	47	**66**	**91**	**116**	**144**	56	**80**	**104**	**131**	**160**
34	**49**	**69**	**93**	**117**	**134**	40	**59**	**82**	**105**	**132**	49	**71**	**94**	**119**	**146**
35	**42**	**61**	**84**	**107**	**127**	33	**51**	**73**	**95**	**120**	41	**62**	**84**	**108**	**134**
36	**36**	**54**	**76**	**97**	**121**		**44**	**65**	**86**	**111**	35	**55**	**75**	**99**	**122**
37	**31**	**47**	**68**	**88**	**113**		**38**	**58**	**77**	**101**		**48**	**67**	**90**	**111**
38		**41**	**61**	**80**	**103**		**32**	**51**	**70**	**92**		**41**	**59**	**81**	**102**

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 25″ c/c Wt 75 psf					6″ Joists @ 26″ c/c Wt 80 psf					7″ Joists @ 27″ c/c Wt 85 psf				
Bottom Bar	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#6	#6	#7	#7	#8
Truss Bar	#6	#6	#7	#7	#8	#6	#6	#7	#7	#8	#6	#7	#7	#8	#8
Top Bar	#5	#6	#6	#7	#6	#5	#6	#6	#7	#6	#6	#6	#7	#6	#8
Span l′ in Ft 21	241	294	315	315	315	224	274				256	317			
22	213	260	291	291	291	197	242	300			225	281			
23	189	232	269	269	269	174	215	267	318	318	199	249	300		
24	**167**	205	250	250	250	153	**190**	238	282	296	175	**222**	263	322	
25	**148**	185	231	233	233	134	**170**	214	258	276	156	**199**	240	289	314
26	**131**	166	207	217	217	118	**152**	191	232	258	138	**176**	215	262	295
27	**116**	148	187	203	203	104	**134**	172	210	242	121	**158**	**194**	237	**277**
28	**103**	**132**	169	191	191	91	**119**	**154**	189	227	107	**141**	**174**	214	**254**
29	**91**	**118**	153	179	179	79	**106**	**139**	171	209	94	**126**	**157**	193	**231**
30	**80**	**106**	137	169	**169**	69	**94**	**124**	154	190	83	**111**	**141**	175	**210**
31	**70**	**94**	124	153	**159**	59	**83**	**112**	140	**173**	72	**99**	**127**	**159**	**191**
32	**61**	**84**	112	139	**149**	51	**73**	**100**	**126**	**157**	62	**88**	**113**	**143**	**175**
33	**54**	**74**	**101**	127	**141**	44	**64**	**89**	**114**	**144**	53	**78**	**102**	**131**	**159**
34	**45**	**66**	**91**	115	**133**	36	**55**	**79**	**103**	**130**	45	**68**	**91**	**118**	**145**
35	**39**	**58**	**81**	104	**126**		**48**	**70**	**92**	**119**	38	**59**	**81**	**107**	**132**
36	**33**	**50**	**72**	94	**120**		**40**	**62**	**83**	**108**	31	**51**	**72**	**96**	**120**
37		**44**	**65**	85	**110**		**34**	**54**	**74**	**98**		**44**	**63**	**87**	**109**
38		**38**	**57**	**77**	**101**			**47**	**66**	**89**		**38**	**56**	**78**	**99**

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—20 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth	14″ FORMS + 3″ CONCRETE														
Joists	5″ Joists @ 25″ c/c Wt 81 psf					6″ Joists @ 26″ c/c Wt 86 psf					7″ Joists @ 27″ c/c Wt 91 psf				
Bottom Bar	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#6	#6	#7	#7	#8
Truss Bar	#6	#6	#7	#7	#8	#6	#6	#7	#7	#8	#6	#7	#7	#8	#8
Top Bar	#5	#6	#6	#7	#6	#5	#6	#6	#7	#6	#6	#6	#7	#6	#8
Span l′ in Ft 21	244	297	321	321	321	227	278				259	321			
22	217	265	297	297	297	200	246	304			229	285			
23	191	235	274	274	274	175	218	271	324		201	253	304		
24	169	209	255	255	255	154	193	242	291	302	177	225	272	327	
25	148	186	233	237	237	135	171	216	261	281	256	200	243	294	321
26	131	166	210	221	221	118	151	194	235	262	137	178	218	265	301
27	116	148	189	206	206	103	135	173	211	246	121	159	195	239	281
28	102	133	170	193	193	90	119	155	191	231	107	141	176	216	257
29	90	118	152	183	183	78	106	138	172	211	93	125	157	195	233
30	78	105	137	170	170	67	93	124	155	192	81	111	141	177	212
31	68	93	123	153	161	58	82	111	139	173	70	98	126	159	193
32	59	83	111	140	151	49	71	99	126	158	61	87	113	144	175
33	51	74	100	127	142	41	63	88	114	144	52	77	102	130	160
34	43	64	89	114	135	34	53	77	102	130	43	66	90	117	145
35	36	56	79	103	127		46	68	91	118	35	57	80	105	131
36	30	48	71	93	120		38	60	82	107		49	70	95	120
37		41	63	84	109		32	52	73	97		42	62	85	108
38		35	55	75	99			45	64	87		35	54	76	98

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth	6″ FORMS + 2½″ CONCRETE							
Joists	4″ Joists @ 34″ c/c Wt 41 psf				5″ Joists @ 35″ c/c Wt 43 psf		6″ Joists @ 36″ c/c Wt 45 psf	
Bottom Bar	#4	#4	#5	#5	#5	#5	#5	#5
Truss Bar	#4	#5	#5	#6	#5	#6	#5	#6
Top Bar	#4	#4	#5	#5	#5	#5	#5	#5
Span l′ in Ft 9	271	271	329	329	336	336	323	323
10	223	239	292	292	298	298	286	286
11	177	215	261	261	267	267	256	256
12	142	191	237	237	227	241	218	231
13	115	156	196	197	187	218	179	210
14	94	129	164	167	156	194	148	186
15	76	107	138	144	131	164	123	156
16	62	89	116	126	109	140	103	129
17	50	74	97	110	91	118	86	112
18	40	61	83	97	77	101	72	95
19	32	51	70	87	65	86	60	80

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 6″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c, Wt 48 psf			5″ Joists @ 35″c/c, Wt 50 psf		6″ Joists @ 36″ c/c, Wt 52 psf	
Bottom Bar	#4	#4	#5	#5	#5	#5	#5
Truss Bar	#4	#5	#5	#5	#6	#5	#6
Top Bar	#4	#4	#5	#5	#5	#5	#5
Span l′ in Ft 9	285	285	347	355	355	342	342
10	234	253	308	314	314	302	302
11	185	224	276	281	281	270	270
12	148	199	247	239	254	229	243
13	119	162	205	196	230	187	220
14	96	133	171	163	206	154	196
15	78	110	143	135	172	128	164
16	62	91	119	113	145	106	138
17	49	75	100	94	123	88	116
18	39	61	84	78	104	73	97
19	30	50	71	65	89	60	83
20		41	59	54	75	49	69

Depth: 8″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c, Wt 45 psf				5″ Joists @ 35″ c/c, Wt 48 psf			6″ Joists @ 36″ c/c, Wt 50 psf		
Bottom Bar	#4	#4	#5	#5	#5	#5	#6	#5	#5	#6
Truss Bar	#4	#5	#5	#6	#5	#6	#6	#5	#6	#6
Top Bar	#4	#5	#5	#5	#5	#5	#6	#5	#5	#6
Span l′ in Ft 10	292	309								
11	233	278			344			331		
12	189	250	306	306	299	312		287	300	
13	154	207	257	257	247	284	307	237	273	333
14	127	172	217	219	207	255	272	197	245	294
15	105	144	183	191	174	216	237	165	207	250
16	86	121	156	166	147	184	208	139	176	212
17	71	102	132	147	124	158	186	117	150	183
18	59	86	113	131	106	136	166	99	128	158
19	48	73	97	117	90	117	144	84	110	136
20	39	61	83	106	77	101	126	71	95	118
21	31	51	71	94	65	87	109	59	81	103
22		43	61	81	55	75	95	50	69	89
23		35	52	71	46	65	83	41	59	77

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 8″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 51 psf				5″ Joists @ 35″c/c Wt 54 psf			6″ Joists @ 36″ c/c Wt 56 psf		
Bottom Bar	#4	#4	#5	#5	#5	#5	#6	#5	#5	#6
Truss Bar	#4	#5	#5	#6	#5	#6	#6	#5	#6	#6
Top Bar	#4	#5	#5	#5	#5	#5	#6	#5	#5	#6
Span l′ in Ft										
10	303	323								
11	243	291								
12	195	260	320	320	310	326		299	311	
13	159	215	267	267	257	296	320	246	284	
14	130	178	225	227	214	268	**284**	204	255	**307**
15	107	148	190	196	180	226	**246**	171	215	**260**
16	**87**	124	160	171	151	192	**217**	143	182	**222**
17	**72**	104	136	151	128	163	**192**	**120**	154	**190**
18	**58**	87	116	134	108	139	**172**	**101**	132	**164**
19	**47**	73	99	**120**	91	120	**149**	**85**	112	**141**
20	**38**	61	84	**108**	**77**	103	**129**	**71**	**97**	**122**
21		**51**	71	**95**	**65**	88	**112**	**59**	**82**	**105**
22		**41**	61	**82**	**54**	76	**98**	**49**	**70**	**91**
23		**34**	51	**71**	**45**	65	**85**	**41**	**59**	**79**

Depth: 10″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 50 psf					5″ Joists @ 35″ c/c Wt 52 psf				6″ Joists @ 36″ c/c Wt 55 psf			
Bottom Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6	#7
Truss Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#6	#6	#7	#7
Top Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#5	#6	#6	#7
Span l′ in Ft													
15	132	180	227	234	234	218	272	293	293	260	311		
16	110	153	194	206	206	185	232	259	259	222	267	**307**	**307**
17	**92**	129	166	182	182	158	200	230	230	189	229	**274**	**274**
18	**76**	110	143	164	164	135	173	207	207	163	199	**243**	**247**
19	**64**	93	123	146	146	116	150	183	**187**	141	173	**212**	**224**
20	**53**	80	106	132	132	**100**	130	160	**170**	**121**	151	**186**	**204**
21	**43**	**68**	91	120	121	**85**	113	140	**155**	**106**	131	**163**	**187**
22	**35**	**57**	78	105	110	**73**	98	124	**142**	**91**	115	**144**	**171**
23		**48**	68	92	**100**	**63**	86	109	**131**	**79**	**101**	**127**	**153**
24		**40**	**58**	80	**92**	**53**	**74**	95	**120**	**68**	**88**	**112**	**137**
25		**33**	**50**	70	**84**	**45**	**65**	84	**106**	**58**	**77**	**99**	**121**
26			**42**	61	**78**	**38**	**56**	74	**95**	**50**	**67**	**88**	**108**
27			**36**	53	**69**	**31**	**48**	64	**85**	**42**	**58**	**78**	**96**
28			**30**	46	**61**		**41**	**56**	**75**	**35**	**50**	**68**	**82**

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 10″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 56 psf					5″ Joists @ 35″ c/c Wt 58 psf					6″ Joists @ 36″ c/c Wt 61 psf			
Bottom Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#6	#6	#7
Truss Bar	#4	#5	#5	#6	#6	#5	#6	#6	#7	#7	#6	#6	#7	#7
Top Bar	#4	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#6	#6	#7
Span l′ in Ft 15	134	184	234	241	241	224	281	302	302	302	268	321		
16	111	155	199	211	211	190	240	266	266	266	229	276	318	318
17	92	131	170	186	186	162	206	236	236	236	195	237	283	283
18	76	111	145	167	167	138	177	213	213	213	167	204	250	255
19	63	94	125	150	150	118	153	188	192	192	144	177	217	231
20	51	79	107	135	135	100	133	164	174	174	124	154	191	210
21	41	67	92	122	122	85	115	143	158	158	107	134	167	192
22	33	56	79	106	111	73	100	125	144	144	92	117	147	176
23		47	68	92	101	62	86	109	132	132	79	102	129	157
24		38	57	80	93	52	74	103	122	122	67	88	113	139
25		30	48	70	84	44	64	83	108	113	58	76	100	122
26			41	60	78	36	55	73	95	105	49	66	88	109
27			34	52	69		47	63	84	96	41	57	77	97
28				44	60		39	55	74	89	34	49	68	86

Depth: 12″ FORMS + 2½″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 54 psf				5″ Joists @ 35″ c/c Wt 57 psf					6″ Joists @ 36″ c/c Wt 61 psf					
Bottom Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#7
Truss Bar	#5	#5	#6	#6	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#8
Top Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#6
Span l′ in Ft 18	134	173	196	196	164	208	247	247	247	153	197	239	293	293	293
19	115	150	177	177	141	181	221	223	223	131	170	209	256	265	265
20	98	130	160	160	122	158	193	204	204	113	148	182	225	242	242
21	84	113	146	146	106	138	170	187	187	97	128	159	199	222	222
22	72	98	129	133	91	121	150	171	171	83	111	140	175	204	204
23	61	85	113	123	78	106	132	157	157	70	97	123	155	187	189
24	52	74	99	112	67	92	117	146	146	60	84	108	138	167	175
25	43	64	87	104	58	80	104	132	135	50	72	95	122	149	162
26	36	55	77	96	49	70	91	117	125	42	62	83	108	133	151
27	30	47	67	87	41	61	80	105	117	35	54	72	96	119	141
28		40	59	77	34	52	71	93	109		45	63	85	106	132
29		34	51	69		45	62	83	101		38	55	75	95	119
30			44	61		38	54	74	93		32	47	66	85	107
31			38	54		33	48	66	84			41	58	76	96
32			32	47			41	58	75			34	51	67	87
33				41			35	51	67				44	59	78

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth: 12″ FORMS + 3″ CONCRETE

Joists	4″ Joists @ 34″ c/c Wt 60 psf				5″ Joists @ 35″ c/c Wt 63 psf					6″ Joists @ 36″ c/c Wt 67 psf					
Bottom Bar	#4	#5	#5	#6	#4	#5	#6	#6	#7	#5	#5	#6	#6	#7	#7
Truss Bar	#5	#5	#6	#6	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#8
Top Bar	#4	#5	#5	#6	#5	#5	#6	#6	#7	#5	#5	#6	#6	#7	#6
Span l′ in Ft 18	135	176	200	200	166	212	253	253	253	155	200	245	300	300	300
19	115	152	179	179	143	184	225	228	228	133	173	213	261	272	272
20	98	131	162	162	123	160	197	207	207	113	149	185	229	248	248
21	83	113	147	148	105	139	172	189	189	96	129	161	201	227	227
22	71	98	130	135	91	122	152	173	173	82	112	141	178	208	208
23	59	85	114	123	78	106	134	159	159	70	97	124	157	189	192
24	50	73	100	113	66	92	117	147	147	58	84	108	139	168	178
25	41	62	87	103	56	80	103	133	137	49	72	94	123	150	165
26	34	53	76	96	47	69	91	117	126	40	61	82	108	133	153
27		45	66	87	39	59	80	104	118	32	52	72	96	119	142
28		37	57	77	32	51	70	93	109		44	62	84	106	132
29		31	49	67		43	61	82	101		36	53	74	94	119
30			42	59		36	53	73	92			45	65	84	107
31			36	52		30	45	64	83			38	56	75	96
32			30	44			39	56	74			32	49	66	86
33				38			33	49	66				42	58	77

Depth: 14″ FORMS + 2½″ CONCRETE

Joists	5″ Joists @ 35″ c/c Wt 62 psf					6″ Joists @ 36″ c/c Wt 66 psf					7″ Joists @ 37″ c/c Wt 70 psf				
Bottom Bar	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#6	#6	#7	#7	#8
Truss Bar	#6	#6	#7	#7	#8	#6	#6	#7	#7	#8	#6	#7	#7	#8	#8
Top Bar	#5	#6	#6	#7	#6	#5	#6	#6	#7	#6	#6	#6	#7	#6	#8
Span l′ in Ft 21	164	201	217	217	217	153	190	235	260	260	179	223	265	299	299
22	144	177	199	199	199	134	163	208	239	239	156	197	236	275	275
23	127	157	184	184	184	117	147	184	221	221	137	174	211	253	256
24	111	138	170	170	170	102	128	164	195	205	119	154	184	227	237
25	97	124	157	161	161	88	115	146	178	191	106	137	167	203	221
26	85	110	140	147	147	77	101	130	159	178	93	121	149	183	206
27	74	97	125	137	137	66	88	116	143	167	80	107	133	164	193
28	65	86	112	128	128	57	78	103	128	156	70	95	119	148	177
29	56	76	100	120	120	49	68	92	115	142	61	84	106	133	160
30	49	67	90	112	112	41	59	81	103	129	52	73	95	120	145
31	41	59	80	101	105	34	51	72	92	116	44	64	84	108	132
32	35	51	71	91	98		44	63	82	105	37	56	75	97	119
33	30	44	63	82	93		37	56	74	96	31	48	66	88	107
34		38	56	74	87		31	49	66	86		42	58	78	98
35		33	49	66	82			42	58	77		35	51	70	88
36			43	59	77			36	51	70			44	62	80
37			38	52	70			31	45	62			38	55	71
38			32	46	63				39	56			33	49	64

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

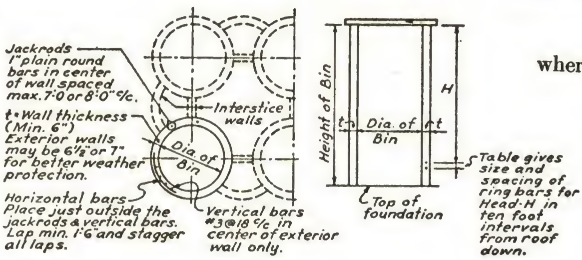
CONCRETE JOIST CONSTRUCTION
INTERIOR SPAN—30 INCH WIDE FORMS
Safe Superimposed Load (psf)

For limitations and explanation of use of tables, see pages 171-173.

Depth	14″ FORMS + 3″ CONCRETE														
Joists	5″ Joists @ 35″ c/c Wt 68 psf					6″ Joists @ 36″ c/c Wt 72 psf					7″ Joists @ 37″ c/c Wt 76 psf				
Bottom Bar	#5	#6	#6	#7	#7	#5	#6	#6	#7	#7	#6	#6	#7	#7	#8
Truss Bar	#6	#6	#7	#7	#8	#6	#6	#7	#7	#8	#6	#7	#7	#8	#8
Top Bar	#5	#6	#6	#7	#6	#5	#6	#6	#7	#6	#6	#6	#7	#6	#8
Span l′ in Ft. 21	164	202	219	219	219	154	191	237	262	262	180	224	269	305	305
22	144	179	202	202	202	134	168	210	243	243	158	198	239	281	281
23	126	158	186	186	186	116	147	186	224	224	138	174	212	256	260
24	110	139	172	172	172	101	129	165	200	208	120	154	189	228	241
25	96	123	156	159	159	87	113	146	179	194	104	136	168	205	224
26	83	108	140	148	148	75	99	130	160	180	90	120	150	183	209
27	72	96	125	137	137	64	87	115	143	167	79	106	133	164	195
28	63	85	111	128	128	55	76	102	128	157	68	93	119	147	177
29	54	74	99	120	120	46	66	90	114	143	59	81	105	132	160
30	46	65	88	111	111	39	57	80	102	128	49	71	93	119	145
31	39	56	78	99	105	32	49	70	91	115	41	62	82	106	130
32	32	49	69	90	98		41	61	82	104	34	54	73	95	118
33		42	62	81	92		35	53	73	91		46	65	85	106
34		35	53	72	84			46	64	84		39	56	75	96
35		30	46	63	80			39	56	75		32	49	67	86
36			40	56	75			33	49	67			42	60	77
37			34	50	68				43	60			36	52	69
38			30	44	60				36	53			31	45	62

Tabulated values in boldface type require embedment E_4 explained on page 171.
Above and to the right of the zigzag line, tapered ends are required.

WALLS FOR CIRCULAR GRAIN STORAGE BINS—INTERNAL PRESSURE



Horiz. radial pressure by Janssen's formula

$$L = \frac{wR}{\mu'}\left[1 - e^{-\left(\frac{k\mu' h}{R}\right)}\right]$$

where L = radial pressure (psf)
w = 50 (pcf) for soybeans, wheat, corn, etc.
$R = \dfrac{\text{area}}{\text{periphery}}$ (ft)
k = 0.60 for soybeans, wheat, corn, etc.
μ' = 0.42 for soybeans, wheat, corn, etc.

Reinforcement is based upon grain (max. weight 50 pcf, min. angle of repose 28°). For heavier material (such as portland cement), increase steel to suit.

Design is for single bin, internally loaded. For cluster, check arch action of bin wall when interstice bin is loaded.

Wall thickness is determined by allowable bearing (540 psi at bottom of wall) while carrying its own weight plus roof, and 80% of total weight of grain in bin. (Min. thickness 6″.)

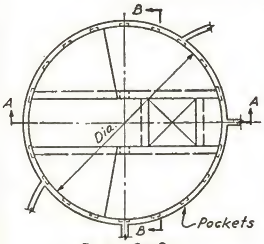
Height of single freestanding circular bin is limited by overturning under 30 psf wind against empty bin and includes all values above zig-zag line in table below. When bins are clustered, height may be increased greatly.

For convenience in placing, specify horizontal bars to lie in same horizontal plane through the entire structure, to supply tiers of bars at uniform vertical spaces for entire height, varying bar sizes to suit.

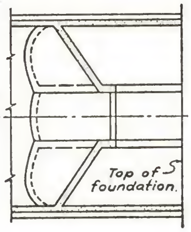
WALL REINFORCEMENT (All walls 6″ min.)

Head H	Inside diameter of bin					
	13′-0	15′-0	18′-0	20′-0	22′-0	24′-0
10	#3 @ 8	#3 @ 8	#3 @ 8	#3 @ 8	#3 @ 8	#3 @ 8
20	#3 @ 8	#3 @ 8	#3 @ 8	#4 @ 12	#4 @ 10	#4 @ 9
30	#3 @ 8	#3 @ 8	#4 @ 12	#4 @ 12	#4 @ 9	#4 @ 8
40	#3 @ 8	#3 @ 8	#4 @ 12	#4 @ 9	#4 @ 8	#4 @ 7
50	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 9	#4 @ 7	#5 @ 9
60	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 11	#5 @ 9
70	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9
80	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9
90	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9
100	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9
110	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9
120	#3 @ 8	#3 @ 8	#4 @ 10	#4 @ 8	#5 @ 10	#5 @ 9

CIRCULAR GRAIN STORAGE BINS—HOPPER BOTTOMS

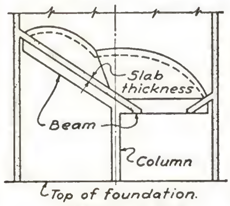


PLAN OF BIN



SECTION B-B

Sometimes grain rests directly on foundation mat, but this necessitates use of power shovel for complete removal. Alternatively, hoppers of structural steel or concrete with sides sloped at least 8-on-12, feeding onto a conveyor system, eliminate shovelling by providing gravity feed.



SECTION A-A

Concrete hoppers are economically constructed as shown in figure, dihedral angles filled in to provide for a slope of at least 8-on-12. Hoppers are supported by pockets in side walls and columns on foundation mat with slabs, beams, and columns designed for the vertical pressures given in the table. Vertical pressure on bin bottoms for conditions given on page 187:—

$$V = \frac{wR}{k\mu'}\left[1 - e^{-\left(\frac{k\mu' h}{R}\right)}\right]$$

VERTICAL PRESSURE ON BIN BOTTOM (psi)

Head H	Diameter of Bins					
	13′-0	15′-0	18′-0	20′-0	22′-0	24′-0
20	507	552	603	627	654	678
30	581	646	728	773	815	852
40	616	695	798	860	916	968
50	630	720	840	913	982	1043
60	638	728	862	942	1020	1095
70	642	738	875	962	1048	1128
80	645	742	883	974	1064	1148
90	645	745	886	983	1072	1164
100	645	745	893	992	1090	1172
110	645	745	893	992	1090	1190
120	645	745	893	992	1090	1190

TWO-WAY FLAT SLABS—SQUARE PANELS WITH DROP PANELS

For two-way dome slabs, see pp. 419 ff.

Tables are presented for the slab thickness, width and thickness of drop panel, strips of reinforcing bars, weight of steel and volume of concrete per square foot of floor area on spans of 15 to 30 ft by one-foot intervals, for safe superimposed loads of 50, 100, 150, 200, 250, 300, 400 and 500 psf for typical square interior panels and for the strips perpendicular to the wall for exterior square panels that are built integrally with a spandrel beam or concrete wall.

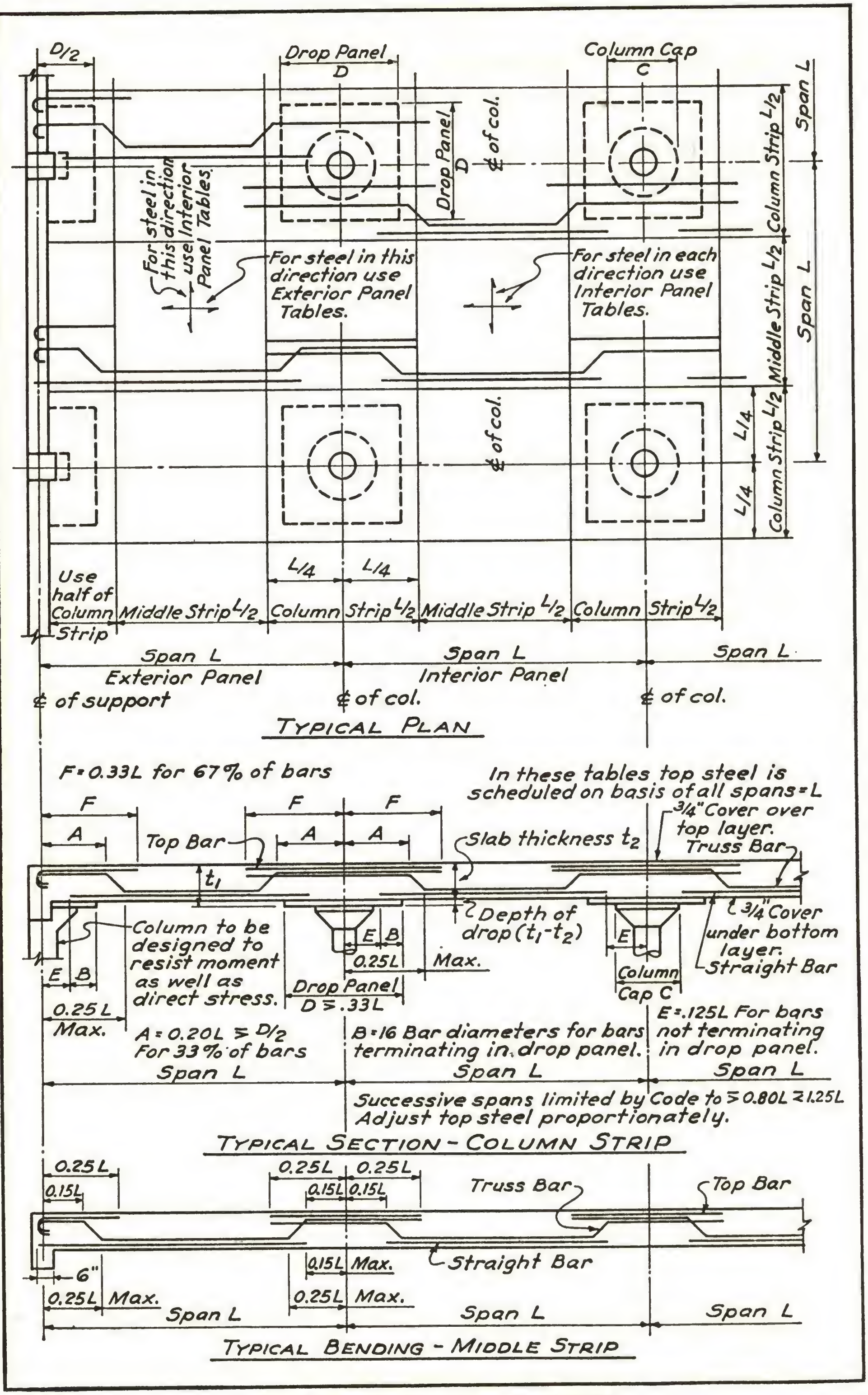
All tables are based upon the recommendations of the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," using one set of fiber stresses, viz., $f'_c = 3000$ psi and $f_s = 20{,}000$ psi, and are based upon using deformed bars whose deformations meet the requirements of ASTM A305.

As shown on page 190, the two-way slab is divided into two bands or strips of reinforcement, one over the columns and extending a quarter panel each side of the center-line, known as a "column strip," and one a half-panel wide between two column strips, known as a "middle strip." A second set of column and middle strips runs at right angles to the first, which explains the designation "two-way" slab. Each strip has straight bars in the bottom and truss bars that are in the bottom at midspan and in the top over the supports, and each strip may, in addition, have supplementary separate top bars when required. The structure must have at least three consecutive panels in a row in each direction to come within the ACI Code values for moments; if the building is narrower, a special analysis must be made which will have the effect of increasing the reinforcement. The successive spans must be of such lengths that they do not differ by more than twenty per cent of the longer span.

While values have been computed only for square panels, it is possible to estimate values for a rectangular panel fairly accurately by using the long side for one set of strips and the short side for the other. The ACI Code limits the ratio of long to short side to 1.33.

For exterior panels, it is possible to take the strips that are continuous, i.e., parallel to the discontinuous edge, from the table for Typical Interior Panels. The strips that are noncontinuous, i.e., perpendicular to the discontinuous edge, are given by the table for Strips Perpendicular to an Exterior Wall, provided that the exterior edge of the panel frames into a concrete column or concrete bearing wall integral with the slab. If the slab simply rests upon a masonry wall without any edge restraint, then the slab and drop panel thicknesses must be increased 15 per cent and the bars in the strips perpendicular to the exterior wall must be changed as follows from the values in the tables for Typical Exterior Panels:—

Positive steel in column strip:—Increase 50%.
Negative steel in column strip at wall:—Decrease to 17% of tabulated value.
Negative steel in column strip over first interior column:—Increase 30%.
Positive steel in middle strip:—Increase 30%.
Negative steel in middle strip at exterior wall:—Decrease to 30% of tabulated value.
Negative steel in middle strip at first interior row of columns:—Increase 33%.


D/2
Drop Panel
D
Column Cap
C
Drop Panel D
℄ of col.
Column Strip L/2
Span L
For steel in this direction use Interior Panel Tables.
For steel in this direction use Exterior Panel Tables.
For steel in each direction use Interior Panel Tables.
Middle Strip L/2
L/4
Use half of Column Strip
Middle Strip L/2
Column Strip L/2
Middle Strip L/2
Column Strip L/2
Span L
Exterior Panel
Interior Panel
℄ of support
℄ of col.
TYPICAL PLAN
F = 0.33L for 67% of bars
In these tables top steel is scheduled on basis of all spans = L
F
A
Top Bar
Slab thickness t_2
3/4" Cover over top layer.
Truss Bar
t_1
Column to be designed to resist moment as well as direct stress.
Depth of drop (t_1-t_2)
3/4" Cover under bottom layer.
Straight Bar
E
B
0.25L
Max.
Drop Panel D = .33L
Column Cap C
0.25 L Max.
A = 0.20L = D/2 For 33% of bars
B = 16 Bar diameters for bars terminating in drop panel.
E = .125L For bars not terminating in drop panel.
Span L
Successive spans limited by Code to = 0.80L = 1.25L
Adjust top steel proportionately.
TYPICAL SECTION - COLUMN STRIP
0.25L
0.15L
Truss Bar
Top Bar
Straight Bar
6"
0.15L Max.
0.25L Max.
Span L
TYPICAL BENDING - MIDDLE STRIP

TWO-WAY FLAT SLABS—SQUARE PANELS WITH DROP PANELS

For corner panels which are discontinuous on two edges, both sets of strips should be taken from the tables for Typical Exterior Panels; if both discontinuous edges rest upon masonry walls, the corrections above shall apply for both sets of strips.

The concrete quantities given per square foot of floor area include all structural concrete in slab and drop panel but do not include any material in the column capital, column, nor any floor finish above the structural slab. Note that the "safe superimposed load" represents live load, floor finishes, partition allowance, and everything except the weight of the concrete. For a table of quantities in columns and column capitals, see page 106.

The weight of steel is the average weight in pounds per sq ft of all bars in the slab but not including bars in beams, columns, walls or footings.

The effective depth of slab is computed on the basis of an allowance of ¾ in. cover over the bars in all cases, and, where bars cross each other, by an allowance of one bar diameter plus 0.03 in. for deformations.

While the scope of these tables is adequate for most purposes, it is not practicable to present all possible combinations. For those who wish to extend beyond the coverage of the tables as well as for those who wish to know how they were computed, the following examples will be instructive:—

Example—For the table on page 196, design a two-way typical interior flat slab panel 20′-0″ square for a safe superimposed load of 250 psf. Stresses:—f_c = 1350 psi, f_s = 20,000 psi. See ACI Code Art. 1002 and 1004.

Column Cap:—Size of cap is determined by keeping stresses around periphery within the allowable,* but 22½ per cent of the panel length (or 4′-6″) is often used and will be tried here. Caps are often made in multiples of 6 in. to suit the standard steel forms available.

Drop Panel:—Size of panel is determined by keeping stresses around periphery within the allowable, but should be at least 0.33 of the panel length in the parallel direction.* (7′-6″ will be assumed here.)

Slab Thickness:—Slab thickness, t_2, shall not be less than $L/40$, which equals 6 in., nor less than $t_2 = 0.024\,L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{f'_c/2000}} + 1$, where c = diameter of column cap in feet, L = span in feet, and w' = uniform dead plus live load, psf, so

$$t_2 = 0.024 \times 20\left(1 - \frac{2 \times 4.5}{3 \times 20}\right)\sqrt{\frac{345}{3000/2000}} + 1'' = 7.28\text{ in.},$$

and shall be sufficient to keep bending and shearing stresses within Code limits. For the present, 7½ in. will be assumed.

Drop Panel Thickness:—ACI 318 requires the total thickness in drop panel to be at least $t_1 = 0.028\,L\left(1 - \frac{2c}{3L}\right)\sqrt{\frac{w'}{f_c'/2000}} + 1\tfrac{1}{2}'' = 0.028 \times 20\left(1 - \frac{2 \times 4.5}{3 \times 20}\right)\sqrt{\frac{345}{3000/2000}}$ $+ 1\tfrac{1}{2}''$ = 8.75 in., and not greater than $1.5t_2$ which is 11¼ in. and is used here.

Total Bending Moment:—$M_o = 0.09\,WLF\left(1 - \frac{2c}{3L}\right)^2$, where $F = 1.15 - \frac{c}{L}$, but $\ngtr 1.00$.

Superimposed load	= 20 × 20 × 250 =	100,000 lb
Slab	= 20 × 20 × 0.64 × 150 =	38,400 lb
Drop	= 7.5 × 7.5 × 0.31 × 150 =	2,600 lb
		141,000 lb

* The 1956 ACI Code is considerably less fixed in the proportions of the parts than previous codes, giving the designer more latitude in selecting outlines, but requiring that the conditions around the periphery of cap or drop and at all critical sections be within Code limits for stresses.

TWO-WAY FLAT SLABS—SQUARE PANELS WITH DROP PANELS

$$F = 1.15 - \frac{4.5}{20} = 0.925 \text{ but must be} \ngtr 1.00.$$

$$M_o = 0.09 \times 141{,}000 \times 20 \times 12 \times 1.00\left(1 - \frac{2 \times 4.5}{3 \times 20}\right)^2 = 2{,}200{,}000 \text{ lb-in.}$$

Column Strip:—Positive moment $= 0.20\ M_o = 440{,}000$ lb-in.

$d = 7.5 - 0.75 - 0.25 = 6.5$ in. Take $b = \frac{3}{4} \times$ panel width (ACI 1002c):—

$$A_s = \frac{M}{jf_sd} = \frac{440{,}000}{\frac{7}{8} \times 20{,}000 \times 6.5} = 3.86 \text{ sq in.*}$$

$$19\text{-\#4 bars} = 3.80 \text{ sq in.}\begin{cases} 9 \text{ Straight} \\ 10 \text{ Truss} \end{cases}$$

$$R = \frac{M}{bd^2} = \frac{440{,}000}{\frac{3}{4} \times 120 \times 6.5 \times 6.5} = 116 < 236, \text{so } f_c < 1350 \text{ psi}$$

Negative moment (computed for a series of equal spans as explained on page 189) = 0.50 M_o = 1,100,000 lb-in.

$d\dagger = 7.5 + 3.75 - 0.75 - 0.50 - 0.03 - 0.25 = 9.72$ in.

Take $b = \frac{3}{4} \times$ drop width (ACI 1002c):—

$$A_s = \frac{M}{jf_sd} = \frac{1{,}100{,}000}{\frac{7}{8} \times 20{,}000 \times 9.72} = 6.47 \text{ sq in.}\ddagger$$

2 × 10-#4 truss bars	=	4.00 sq in.
12-#4 top bars	=	2.40 sq in.
		6.40 sq in.

$$R = \frac{M}{bd^2} = \frac{1{,}100{,}000}{\frac{3}{4} \times 90 \times 9.72 \times 9.72} = 172 < 236, \text{so } f_c < 1350 \text{ psi}$$

Middle Strip:—Positive moment $= 0.15\ M_o = 330{,}000$ lb-in.

$d = 7.5 - 0.75 - 0.5 - 0.03 - 0.25 = 5.97$, since the bars in the middle strip in one direction must rest on top of those at right angles, thus reducing the effective depth.

$$A_s = \frac{330{,}000}{\frac{7}{8} \times 20{,}000 \times 5.97} = 3.16 \text{ sq in.}\S \quad 15\text{-\#4 bars} = 3.00 \text{ sq in.}\begin{cases} 7 \text{ Straight} \\ 8 \text{ Truss} \end{cases}$$

Since the negative moment has the same numerical value and since the bars are all in one layer at the top and have a better moment arm, it is unnecessary to recompute the moment here; simply bend up one-half or slightly over one-half of the bars in this strip.

Shear around Column Cap:—This is figured on a vertical section around a circle with a radius that is larger than the cap radius by the total thickness of slab and drop panel less 1½ in. (ACI 1002c2):—

$$v = \frac{V}{bjd} = \frac{(20 \times 20 - \pi \times 3.06 \times 3.06) \times 344}{2 \times \pi \times 36\frac{3}{4} \times \frac{7}{8} \times 9.72} = 66 \text{ psi} < 90 \text{ psi}$$

---

* The area furnished is about 1½ per cent less than that computed as necessary assuming $j = \frac{7}{8}$. However, since $p = 0.0065$, based on three-fourths of the width of column band, $j = 0.900$ (page 34) and $A_s = 3.76$ sq in., so that the proposed steel is sufficient.

† This equals slab thickness plus drop thickness less fireproofing less top layer of bars (as lower layer has less moment arm) less 0.03 for deformations and less half the diameter of the lower layer of top bars.

‡ The area provided is about 1 per cent less than the computed area assuming $j = \frac{7}{8}$. However, since $p = 0.00975$, based on three-fourths of the width of drop panel, $j = 0.882$ (page 34) and $A_s = 6.41$ sq in., so that the proposed steel is sufficiently close to the requirements.

§ Checking again, $p = 0.0056$, $j = 0.906$, $A_s = 3.05$ sq in., so that the 3.00 sq in. furnished is 1⅔ per cent on the low side in the upper layer of bars. It would, of course, be simple enough to increase the steel, but with bar weights permitted to vary 3.5 per cent (page 402) and with the permissible tolerances of concrete construction, exact precision seems out of place.

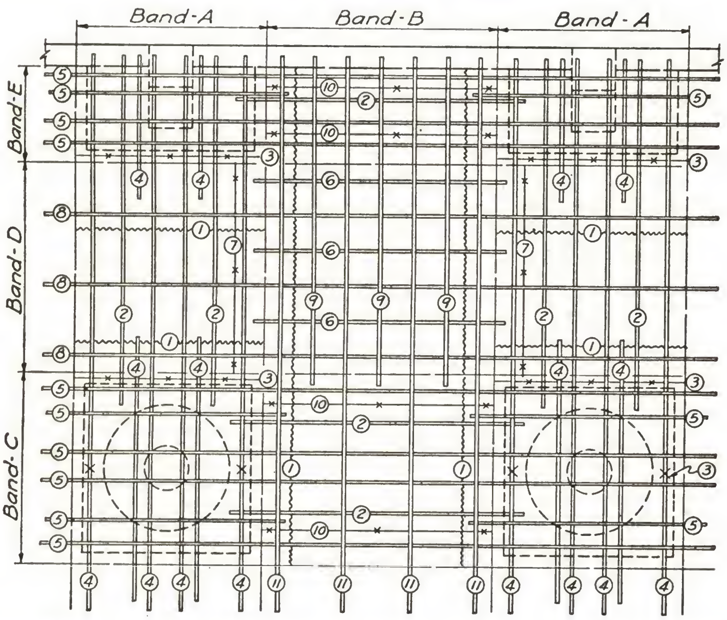
TWO-WAY FLAT SLABS—SQUARE PANELS WITH DROP PANELS

Shear around Drop Panel:—This is figured on a vertical section around a square, each side of which lies at a distance beyond the drop panel equal to the slab thickness less $1\frac{1}{2}$ in. (ACI 1002c3):—

$$v = \frac{V}{bjd} = \frac{(20 \times 20 - 8.5 \times 8.5) \times 344}{4 \times 102 \times \frac{7}{8} \times 5.97} = 52.7 \text{ psi} < 90 \text{ psi}$$

It is impracticable to present a full study of flat slabs in a manual such as this or to tabulate more than a few typical cases. The negative moment and top steel in these tables is computed for a series of equal spans. In those cases where the adjoining span is longer or shorter than the span under consideration, adjustment must be made in the amount of top steel. Rectangular panels, openings through slabs, spans varying more than 20 per cent, panels with marginal beams or with interior beams, and many similar variations require special treatment best undertaken by a structural engineer.

Two-Way Flat Slab Placing Instructions

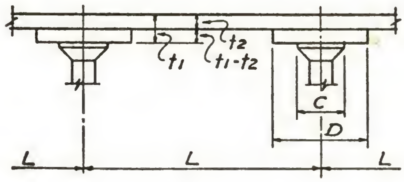


To eliminate placing bars underneath bands already laid, install bars and chairs in the sequence indicated by the numbers given in order below and shown in circles on the layout:—

1. Place lower slab bar spacers—shown ~~~~~~~.
2. Place straight bars only—Bands A, C and E.
3. Place (close to drop) support bars and individual high chairs—Bands A—shown –x—x—x–.
4. Place bent bars and top bars—Bands A.
5. Place bent bars and top bars—Bands C and E.
6. Place straight bars only—Band D.
7. Place support bars and individual high chairs—Band D—shown –x—x—x–.
8. Place bent bars—Band D.
9. Place straight bars only—Band B.
10. Place support bars and individual high chairs—Band B—shown –x—x—x–.
11. Place bent bars—Band B.

READY FOR CONCRETE.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Super-imposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant.	Trussed Bar No.	Trussed Length
L = 15′-0	50	4¾	2½	5	#3	11′-9	4	#3	25′-9
	100	5	2½	5	#3	11′-9	6	#3	25′-9
	150	5½	2¾	6	#3	11′-9	7	#3	25′-9
D = 5′-6	200	5¾	3	4	#4	11′-9	5	#4	25′-9
	250	6	3	5	#4	11′-9	5	#4	25′-9
	300	6¼	3¼	5	#4	11′-9	6	#4	25′-9
C = 3′-0	400	6¾	3¼	6	#4	11′-9	7	#4	26′-0
	500	8	3½	6	#4	11′-9	7	#4	26′-0
L = 16′-0	50	5	2½	5	#3	12′-6	5	#3	27′-6
	100	5½	2¾	6	#3	12′-6	7	#3	27′-6
	150	5¾	3	8	#3	12′-6	8	#3	27′-6
D = 6′-0	200	6	3	5	#4	12′-6	5	#4	27′-6
	250	6¼	3¼	6	#4	12′-6	6	#4	27′-6
	300	6½	3¼	6	#4	12′-6	7	#4	27′-6
C = 3′-6	400	7	3½	7	#4	12′-6	8	#4	27′-9
	500	8	3¾	5	#5	12′-6	5	#5	27′-9
L = 17′-0	50	5½	3	5	#3	13′-0	6	#3	29′-0
	100	5¾	3	7	#3	13′-0	8	#3	29′-0
	150	6	3	5	#4	13′-0	5	#4	29′-0
D = 6′-4	200	6	3	6	#4	13′-0	7	#4	29′-0
	250	6½	3¼	7	#4	13′-0	7	#4	29′-0
	300	7	3½	7	#4	13′-0	8	#4	29′-0
C = 3′-6	400	7½	3¾	5	#5	13′-0	6	#5	29′-0
	500	8	4	6	#5	13′-0	6	#5	29′-0
L = 18′-0	50	6	3	6	#3	13′-9	6	#3	30′-9
	100	6	3	8	#3	13′-9	9	#3	30′-9
	150	6¼	3½	6	#4	13′-9	6	#4	30′-9
D = 6′-9	200	6½	3¼	7	#4	13′-9	7	#4	30′-9
	250	6¾	3½	8	#4	13′-9	8	#4	30′-9
	300	7¼	3¾	8	#4	13′-9	9	#4	30′-9
C = 4′-0	400	8	4	6	#5	13′-9	6	#5	30′-9
	500	8½	4	7	#5	13′-9	7	#5	30-′9

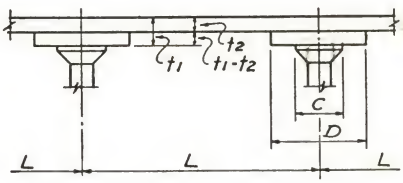
TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS

f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip						Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor*
Top			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
6	#3	10′-0	4	#3	10′-6	5	#3	22′-9	1.15	.425
6	#3	10′-0	4	#3	10′-6	5	#3	22′-9	1.74	.446
7	#3	10′-0	5	#3	10′-6	6	#3	22′-9	1.83	.491
4	#4	10′-0	6	#3	10′-6	7	#3	22′-9	2.13	.514
6	#4	10′-0	7	#3	10′-6	7	#3	22′-9	2.34	.532
7	#4	10′-0	4	#4	10′-6	5	#4	22′-9	2.63	.560
8	#4	10′-0	5	#4	10′-6	6	#4	23′-0	2.83	.604
9	#4	10′-0	5	#4	10′-6	5	#4	23′-0	3.03	.710
5	#3	10′-9	5	#3	11′-3	5	#3	24′-3	1.16	.446
6	#3	10′-9	5	#3	11′-3	5	#3	24′-3	1.42	.491
8	#3	10′-9	6	#3	11′-3	6	#3	24′-3	1.79	.514
7	#4	10′-9	7	#3	11′-3	7	#3	24′-3	2.13	.534
8	#4	10′-9	8	#3	11′-3	8	#3	24′-3	2.44	.560
7	#4	10′-9	5	#4	11′-3	5	#4	24′-3	2.67	.580
9	#4	10′-9	6	#4	11′-3	6	#4	24′-6	3.20	.625
8	#5	10′-9	6	#4	11′-3	7	#4	24′-6	3.59	.693
6	#3	11′-3	5	#3	12′-0	5	#3	25′-9	1.25	.494
8	#3	11′-3	5	#3	12′-0	6	#3	25′-9	1.62	.515
7	#4	11′-3	7	#3	12′-0	7	#3	25′-9	2.03	.535
6	#4	11′-3	8	#3	12′-0	9	#3	25′-9	2.46	.535
8	#4	11′-3	9	#3	12′-0	9	#3	25′-9	2.65	.580
8	#4	11′-3	5	#4	12′-0	6	#4	26′-0	2.90	.625
6	#5	11′-3	6	#4	12′-0	7	#4	26′-0	3.36	.669
8	#5	11′-3	7	#4	12′-0	8	#4	26′-0	3.72	.714
7	#3	12′-0	5	#3	12′-9	5	#3	27′-3	1.28	.535
8	#3	12′-0	6	#3	12′-9	7	#3	27′-3	1.74	.535
8	#4	12′-0	5	#4	12′-9	5	#4	27′-3	2.26	.560
8	#4	12′-0	5	#4	12′-9	6	#4	27′-3	2.60	.580
8	#4	12′-0	6	#4	12′-9	6	#4	27′-3	2.78	.604
9	#4	12′-0	6	#4	12′-9	7	#4	27′-3	3.06	.648
8	#5	12′-0	7	#4	12′-9	8	#4	27′-3	3.66	.714
9	#5	12′-0	8	#4	12′-9	9	#4	27′-3	4.12	.755

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Super-imposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant.	Trussed Bar No.	Trussed Length
L = 19′-0	50	6	3	7	#3	14′-3	7	#3	32′-3
	100	6¼	3½	9	#3	14′-3	10	#3	32′-3
	150	6½	3½	7	#4	14′-3	8	#4	32′-3
D = 7′-3	200	7	3½	7	#4	14′-3	8	#4	32′-6
	250	7¼	3¾	6	#4	14′-3	9	#4	32′-6
	300	7¾	4	6	#5	14′-6	6	#4	32′-6
C = 4′-0	400	8½	4¼	7	#5	14′-6	7	#5	32′-6
	500	9	4½	7	#5	14′-6	8	#5	32′-6
L = 20′-0	50	6½	3¼	7	#3	15′-0	9	#3	34′-3
	100	6¾	3½	10	#3	15′-0	11	#3	34′-3
	150	7	3½	7	#4	15′-0	8	#4	34′-3
D = 7′-6	200	7	3½	8	#4	15′-0	10	#4	34′-3
	250	7½	3¾	9	#4	15′-0	10	#4	34′-3
	300	8	4	10	#4	15′-0	11	#4	34′-3
C = 4′-6	400	9	4½	7	#5	15′-0	8	#5	34′-6
	500	10	5	8	#5	15′-0	8	#5	34′-6
L = 21′-0	50	7	3½	8	#3	15′-9	10	#3	36′-0
	100	7	3½	7	#4	15′-9	7	#4	36′-0
	150	7	3½	8	#4	15′-9	9	#4	36′-0
D = 7′-9	200	7½	3¾	9	#4	15′-9	10	#4	36′-0
	250	8	4	7	#5	15′-9	7	#5	36′-0
	300	8½	4¼	7	#5	15′-9	8	#5	36′-0
C = 4′-6	400	9½	4¾	8	#5	15′-9	9	#5	36′-0
	500	10½	5¼	9	#5	15′-9	9	#5	36′-0
L = 22′-0	50	7	3½	10	#3	16′-3	10	#3	37′-9
	100	7	3½	8	#4	16′-3	8	#4	37′-9
	150	7	3½	10	#4	16′-3	10	#4	37′-9
D = 8′-3	200	7½	3¾	7	#5	16′-3	7	#5	37′-9
	250	8	4	8	#5	16′-3	8	#5	37′-9
	300	8½	4¼	8	#5	16′-3	9	#5	37′-9
C = 5′-0	400	9½	4¾	9	#5	16′-3	10	#5	37′-9
	500	10½	5¼	10	#5	16′-3	11	#5	37′-9

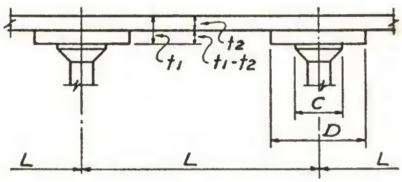
TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS

f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip						Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor*
Top			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
9	#3	12′-9	5	#3	13′-6	6	#3	28′-9	1.42	.535
10	#3	12′-9	7	#3	13′-6	8	#3	28′-9	1.85	.562
8	#4	12′-9	6	#4	13′-6	6	#4	28′-9	2.57	.584
9	#4	12′-9	6	#4	13′-6	6	#4	29′-0	2.67	.625
11	#4	12′-9	7	#4	13′-6	7	#4	29′-0	3.04	.650
9	#5	12′-9	7	#4	13′-6	8	#4	29′-0	3.38	.693
9	#5	12′-9	5	#5	13′-6	6	#5	29′-0	3.90	.758
10	#5	12′-9	6	#5	13′-6	6	#5	29′-0	4.30	.825
8	#3	13′-3	6	#3	14′-0	7	#3	30′-3	1.55	.580
12	#3	13′-3	8	#3	14′-0	9	#3	30′-3	2.04	.605
8	#4	13′-3	10	#3	14′-0	10	#3	30′-3	2.49	.625
9	#4	13′-3	7	#4	14′-0	7	#4	30′-3	3.00	.625
12	#4	13′-3	7	#4	14′-0	8	#4	30′-3	3.24	.671
14	#4	13′-3	8	#4	14′-0	9	#4	30′-3	3.67	.713
9	#5	13′-3	6	#5	14′-0	6	#5	30′-6	4.00	.805
12	#5	13′-3	6	#5	14′-0	7	#5	30′-6	4.61	.885
9	#3	14′-0	7	#3	15′-0	7	#3	32′-0	1.62	.625
8	#4	14′-0	5	#4	15′-0	6	#4	32′-0	2.29	.625
10	#4	14′-0	7	#4	15′-0	7	#4	32′-0	2.81	.625
11	#4	14′-0	7	#4	15′-0	8	#4	32′-0	3.10	.670
8	#5	14′-0	8	#4	15′-0	9	#4	32′-0	3.50	.716
8	#5	14′-0	6	#5	15′-0	6	#5	32′-0	3.75	.760
10	#5	14′-0	6	#5	15′-0	7	#5	32′-0	4.30	.850
12	#5	14′-0	7	#5	15′-0	7	#5	32′-0	4.55	.931
13	#3	14′-9	8	#3	15′-6	8	#3	33′-6	1.60	.625
9	#4	14′-9	6	#4	15′-6	6	#4	33′-6	2.30	.625
12	#4	14′-9	7	#4	15′-6	8	#4	33′-6	3.04	.625
10	#5	14′-9	5	#5	15′-6	6	#5	33′-6	3.50	.670
12	#5	14′-9	6	#5	15′-6	7	#5	33′-6	4.07	.715
10	#5	14′-9	6	#5	15′-6	7	#5	33′-6	4.12	.760
12	#5	14′-9	7	#5	15′-6	8	#5	33′-6	4.70	.842
13	#5	14′-9	8	#5	15′-6	8	#5	33′-6	5.01	.931

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Super-imposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip					
				Straight			Trussed		
				Quant.	Bar No.	Length	Quant.	Bar No.	Length
L = 23′-0	50	7¼	3¾	11	#3	17′-0	12	#3	39′-3
	100	7½	3¾	8	#4	17′-0	9	#4	39′-3
	150	7½	3¾	10	#4	17′-0	11	#4	39′-3
D = 8′-0	200	8	4	7	#5	17′-0	8	#5	39′-6
	250	8½	4¼	8	#5	17′-0	9	#5	39′-6
	300	9	4½	6	#6	17′-6	7	#6	39′-6
C = 5′-0	400	10	5	7	#6	17′-6	8	#6	39′-6
	500	12	5	7	#6	17′-6	8	#6	39′-6
L = 24′-0	50	7½	3¾	7	#4	17′-6	8	#4	41′-0
	100	7½	3¾	9	#4	17′-6	10	#4	41′-0
	150	8	4	7	#5	17′-6	8	#5	41′-0
D = 8′-9	200	8½	4½	8	#5	17′-6	9	#5	41′-0
	250	9	4½	6	#6	18′-0	7	#6	41′-0
	300	9½	4¾	7	#6	18′-0	7	#6	41′-0
C = 5′-6	400	11	4¾	7	#6	18′-0	8	#6	41′-0
	500	13	4¾	8	#6	18′-0	8	#6	41′-3
L = 25′-0	50	7¾	4	8	#4	18′-3	8	#4	43′-0
	100	8	4	10	#4	18′-3	10	#4	43′-0
	150	8½	4¼	8	#5	18′-3	8	#5	43′-0
D = 9′-3	200	9	4½	9	#5	18′-3	9	#6	43′-0
	250	9½	4¾	7	#6	18′-3	7	#6	43′-0
	300	10	5	7	#6	18′-3	8	#6	43′-0
C = 5′-6	400	11	5½	8	#6	18′-3	9	#6	43′-3
	500	13	5½	9	#6	18′-3	9	#6	43′-3
L = 26′-0	50	8¼	4¼	8	#4	19′-0	9	#4	44′-6
	100	8½	4¼	7	#5	19′-0	8	#5	44′-6
	150	8¾	4½	9	#5	19′-0	9	#5	44′-6
D = 9′-6	200	9	4¾	10	#5	19′-6	10	#5	44′-9
	250	9½	4¾	8	#6	19′-6	8	#6	44′-9
	300	10½	5	8	#6	19′-6	9	#6	44′-9
C = 5′-6	400	11½	5¾	9	#6	20′-0	10	#6	45′-0
	500	13½	6	9	#6	20′-0	10	#6	45′-0

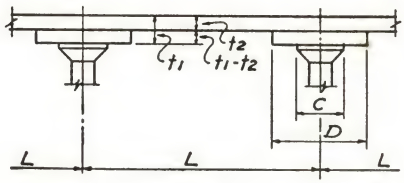
TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS

f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip						Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor*
Top			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
13	#3	15′-3	9	#3	16′-3	9	#3	35′-0	1.88	.650
9	#4	15′-3	6	#4	16′-3	7	#4	35′-0	2.54	.670
12	#4	15′-3	8	#4	16′-3	9	#4	35′-0	3.18	.670
10	#5	15′-3	6	#5	16′-3	6	#5	35′-3	3.60	.714
10	#5	15′-3	6	#5	16′-3	7	#5	35′-3	4.00	.760
8	#6	15′-3	7	#5	16′-3	7	#5	35′-3	4.40	.803
9	#6	15′-3	8	#5	16′-3	8	#5	35′-3	4.98	.891
11	#6	15′-3	8	#5	16′-3	8	#5	35′-3	5.12	1.060
7	#4	16′-0	6	#4	17′-0	6	#4	36′-6	2.05	.670
11	#4	16′-0	7	#4	17′-0	8	#4	36′-6	2.64	.670
8	#5	16′-0	6	#5	17′-0	6	#5	36′-6	3.21	.714
11	#5	16′-0	6	#5	17′-0	7	#5	36′-6	3.66	.760
8	#6	16′-0	7	#5	17′-0	7	#5	36′-6	4.01	.803
10	#6	16′-0	8	#5	17′-0	8	#5	36′-6	4.46	.847
11	#6	16′-0	8	#5	17′-0	9	#5	36′-6	4.95	.972
13	#6	16′-0	8	#5	17′-0	9	#5	36′-9	5.25	1.139
10	#4	16′-6	6	#4	17′-6	6	#4	38′-0	2.11	.692
13	#4	16′-6	8	#4	17′-6	8	#4	38′-0	2.75	.715
10	#5	16′-6	6	#5	17′-6	7	#5	38′-0	3.45	.758
11	#5	16′-6	7	#5	17′-6	7	#5	38′-0	3.76	.803
9	#6	16′-6	5	#6	17′-6	6	#6	38′-0	4.29	.847
9	#6	16′-6	6	#6	17′-6	6	#6	38′-0	4.57	.892
10	#6	16′-6	7	#6	17′-6	7	#6	38′-3	5.21	.982
13	#6	16′-6	7	#6	17′-6	7	#6	38′-3	5.57	1.148
10	#4	17′-3	7	#4	18′-3	7	#4	39′-6	2.23	.714
9	#5	17′-3	6	#5	18′-3	6	#5	39′-6	3.02	.717
11	#5	17′-3	7	#5	18′-3	7	#5	39′-6	3.55	.760
13	#5	17′-3	8	#5	18′-3	8	#5	39′-6	4.04	.805
10	#6	17′-3	9	#5	18′-3	9	#5	40′-0	4.65	.848
9	#6	17′-3	9	#5	18′-3	9	#5	40′-0	4.76	.934
11	#6	17′-3	7	#6	18′-3	8	#6	40′-0	5.59	1.026
13	#6	17′-3	7	#6	18′-3	8	#6	40′-0	5.78	1.195

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Super-imposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant. No.	Trussed Bar No.	Trussed Length
L = 27′-0	50	8½	4¼	9	#4	19′-6	10	#4	46′-0
	100	9	4½	8	#5	19′-6	8	#5	46′-0
	150	9	4½	9	#5	19′-6	10	#5	46′-0
D = 10′-0	200	9½	4¾	8	#6	20′-0	8	#6	46′-3
	250	10	5	8	#6	20′-0	9	#6	46′-3
	300	10½	5¼	9	#6	20′-0	10	#6	46′-3
C = 6′-0	400	12	6	10	#6	20′-0	11	#6	46′-3
	500	13½	6	8	#7	21′-0	8	#7	46′-6
L = 28′-0	50	8½	4¼	10	#4	20′-3	11	#4	48′-0
	100	9	4½	8	#5	20′-3	9	#5	48′-0
	150	9	4¾	12	#5	20′-3	11	#5	48′-0
D = 10′-6	200	10	5	13	#5	20′-3	12	#5	48′-3
	250	10½	5¼	10	#6	20′-6	10	#6	48′-3
	300	11½	5½	10	#6	20′-6	10	#6	48′-3
C = 6′-0	400	12½	6	11	#6	20′-6	12	#6	48′-6
	500	14	7	8	#7	21′-0	9	#7	48′-6
L = 29′-0	50	8¾	4½	7	#5	21′-0	8	#5	49′-9
	100	9¼	4¾	9	#5	21′-0	10	#5	49′-9
	150	9½	5	11	#5	21′-0	13	#5	49′-9
D = 11′-0	200	10	5	10	#6	21′-0	10	#6	49′-9
	250	11	5½	10	#6	21′-0	10	#6	50′-6
	300	11½	5½	10	#6	21′-0	13	#6	50′-6
C = 6′-6	400	13	6¼	12	#6	21′-0	13	#6	50′-9
	500	14½	7	9	#7	21′-6	10	#7	50′-9
L = 30′-0	50	9	4½	8	#5	21′-6	9	#5	51′-6
	100	9½	4¾	10	#5	21′-6	11	#5	51′-6
	150	10	5	8	#6	22′-0	9	#6	51′-6
D = 11′-3	200	10½	5¼	10	#6	22′-0	11	#6	51′-6
	250	11½	5½	11	#6	22′-0	11	#6	51′-6
	300	12	5¾	9	#7	23′-0	9	#7	51′-6
C = 6′-6	400	13½	6½	9	#7	23′-0	10	#7	51′-9
	500	15	7	10	#7	23′-0	10	#7	52′-0

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—INTERIOR PANELS

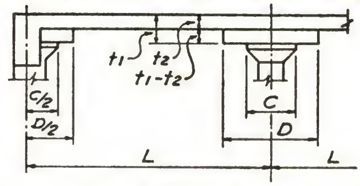
f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip						Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor*
Top			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
12	#4	18′-0	7	#4	19′-0	8	#4	41′-0	2.41	.769
10	#5	18′-0	9	#4	19′-0	10	#4	41′-0	3.15	.803
11	#5	18′-0	11	#4	19′-0	12	#4	41′-0	3.72	.803
10	#6	18′-0	8	#5	19′-0	9	#5	41′-0	4.47	.846
11	#6	18′-0	9	#5	19′-0	10	#5	41′-0	4.88	.892
11	#6	18′-0	10	#5	19′-0	10	#5	41′-0	5.19	.937
13	#6	18′-0	9	#6	19′-0	8	#6	41′-0	5.86	1.072
12	#7	18′-0	9	#6	19′-0	9	#6	41′-3	6.47	1.195
13	#4	18′-6	8	#4	19′-9	9	#4	42′-9	2.58	.758
11	#5	18′-6	10	#4	19′-9	11	#4	42′-9	3.23	.802
12	#5	18′-6	14	#4	19′-9	15	#4	42′-9	3.87	.844
18	#5	18′-6	15	#4	19′-9	16	#4	42′-9	4.50	.934
13	#6	18′-6	11	#5	19′-9	12	#5	42′-9	4.90	1.021
15	#6	18′-6	11	#5	19′-9	12	#5	43′-0	5.41	1.064
15	#6	18′-6	12	#5	19′-9	13	#5	43′-0	6.13	1.153
11	#7	18′-6	9	#6	19′-9	9	#6	43′-0	6.38	1.246
9	#5	19′-3	9	#4	20′-6	9	#4	44′-3	2.70	.781
11	#5	19′-3	11	#4	20′-6	12	#4	44′-3	3.49	.805
14	#5	19′-3	15	#4	20′-6	16	#4	44′-3	4.19	.891
13	#6	19′-3	11	#5	20′-6	11	#5	44′-3	4.73	.934
14	#6	19′-3	11	#5	20′-6	12	#5	44′-6	5.27	.980
13	#6	19′-3	12	#5	20′-6	13	#5	44′-6	5.56	1.063
16	#6	19′-3	14	#5	20′-6	14	#5	44′-9	6.44	1.196
12	#7	19′-3	10	#6	20′-6	10	#6	44′-9	6.86	1.288
9	#5	20′-0	10	#4	21′-0	10	#4	45′-6	2.88	.803
12	#5	20′-0	12	#4	21′-0	12	#4	45′-6	3.66	.846
11	#6	20′-0	9	#5	21′-0	10	#5	45′-6	4.49	.892
13	#6	20′-0	12	#5	21′-0	12	#5	45′-6	5.10	.978
16	#6	20′-0	12	#5	21′-0	13	#5	45′-6	5.69	1.062
12	#7	20′-0	9	#6	21′-0	10	#6	45′-6	6.34	1.066
12	#7	20′-0	10	#6	21′-0	11	#6	45′-9	6.84	1.141
15	#7	20′-0	10	#6	21′-0	11	#6	45′-9	7.15	1.330

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS— BANDS PERPENDICULAR TO AN EXTERIOR WALL

For Bands Parallel to Walls or Not in End Spans, Use "Two-Way Flat Slab Floors, Square Panels—Interior Panels," Pages 194 to 201.



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Super-imposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip								
				Straight			Trussed			Top at Ext. Col.		
				Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length
	50	4¾	2½	5	#3	13′-6	5	#3	19′-6	7	#3	5′-6
L = 15′-0	100	5	2½	7	#3	13′-6	7	#3	19′-6	8	#3	5′-6
	150	5½	2¾	8	#3	13′-6	9	#3	19′-6	8	#3	5′-6
	200	5¾	3	5	#4	13′-6	6	#4	19′-6	7	#4	5′-6
D = 5′-6	250	6	3	6	#4	13′-6	7	#4	19′-9	7	#4	5′-6
	300	6¼	3¼	7	#4	13′-6	7	#4	19′-9	8	#4	5′-6
C = 3′-0	400	6¾	3¼	8	#4	13′-6	8	#4	19′-9	9	#4	5′-6
	500	8	3½	8	#4	13′-6	8	#4	19′-9	10	#4	5′-6
	50	5	2½	6	#3	14′-3	6	#3	21′-0	7	#3	5′-9
	100	5½	2¾	8	#3	14′-3	8	#3	21′-0	8	#3	5′-9
L = 16′-0	150	5¾	3	9	#3	14′-3	10	#3	21′-0	10	#3	5′-9
	200	6	3	6	#4	14′-3	7	#4	21′-0	7	#4	5′-9
D = 6′-0	250	6¼	3¼	7	#4	14′-3	8	#4	21′-0	8	#4	5′-9
	300	6½	3¼	8	#4	14′-3	8	#4	21′-0	9	#4	5′-9
C = 3′-6	400	7	3½	8	#4	14′-3	9	#4	21′-0	10	#4	5′-9
	500	8	3¾	6	#5	14′-3	7	#5	21′-0	8	#5	5′-9
	50	5½	3	6	#3	15′-0	7	#3	22′-0	8	#3	6′-3
L = 17′-0	100	5¾	3	9	#3	15′-0	9	#3	22′-0	10	#3	6′-3
	150	6	3	6	#4	15′-0	7	#4	22′-0	7	#4	6′-3
D = 6′-4	200	6	3	7	#4	15′-0	8	#4	22′-0	8	#4	6′-3
	250	6½	3¼	8	#4	15′-0	9	#4	22′-0	9	#4	6′-3
	300	7	3½	9	#4	15′-0	9	#4	22′-3	11	#4	6′-3
C = 3′-6	400	7½	3¾	7	#5	15′-0	7	#5	22′-3	8	#5	6′-3
	500	8	4	7	#5	15′-0	8	#5	22′-6	8	#5	6′-3
	50	6	3	7	#3	16′-0	8	#3	23′-6	8	#3	6′-6
L = 18′-0	100	6	3	10	#3	16′-0	11	#3	23′-6	11	#3	6′-6
	150	6¼	3¼	7	#4	16′-0	8	#4	23′-9	8	#4	6′-6
D = 6′-9	200	6½	3¼	8	#4	16′-0	9	#4	23′-9	9	#4	6′-6
	250	6¾	3½	9	#4	16′-0	10	#4	23′-9	10	#4	6′-6
C = 4′-0	300	7¼	3¾	10	#4	16′-0	11	#4	23′-9	12	#4	6′-6
	400	8	4	7	#5	16′-0	8	#5	23′-9	8	#5	6′-6
	500	8½	4	8	#5	16′-0	9	#5	23′-9	10	#5	6′-6

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—BANDS PERPENDICULAR TO AN EXTERIOR WALL

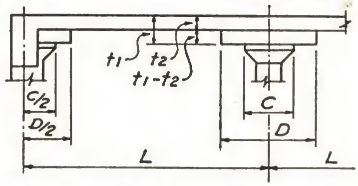
f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip									Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor *
Top at Int. Col.			Top at Ext. Col.			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
6	#3	10′-0	5	#3	4′-3	5	#3	13′-0	5	#3	19′-6	1.25	.425
6	#3	10′-0	6	#3	4′-3	5	#3	13′-0	6	#3	19′-6	1.64	.446
7	#3	10′-0	7	#3	4′-3	6	#3	13′-0	7	#3	19′-6	1.94	.491
6	#4	10′-0	8	#3	4′-3	8	#3	13′-0	8	#3	19′-6	2.30	.514
6	#4	10′-0	9	#3	4′-3	8	#3	13′-0	9	#3	19′-6	2.59	.532
7	#4	10′-0	6	#4	4′-3	5	#4	13′-0	6	#4	19′-6	2.90	.560
9	#4	10′-0	7	#4	4′-3	6	#4	13′-0	7	#4	19′-6	3.46	.604
10	#4	10′-0	7	#4	4′-3	6	#4	13′-0	7	#4	19′-9	3.49	.710
6	#3	10′-9	5	#3	4′-6	5	#3	13′-9	5	#3	20′-9	1.28	.446
7	#3	10′-9	6	#3	4′-6	6	#3	13′-9	6	#3	20′-9	1.70	.491
9	#3	10′-9	8	#3	4′-6	7	#3	13′-9	8	#3	20′-9	2.07	.514
7	#4	10′-9	9	#3	4′-6	9	#3	13′-9	9	#3	20′-9	2.49	.534
8	#4	10′-9	6	#4	4′-6	6	#4	13′-9	6	#4	20′-9	2.86	.560
9	#4	10′-9	7	#4	4′-6	6	#4	13′-9	7	#4	20′-9	3.17	.580
9	#4	10′-9	8	#4	4′-6	7	#4	13′-9	8	#4	20′-9	3.59	.625
7	#5	10′-9	8	#4	4′-6	8	#4	13′-9	8	#4	21′-0	4.07	.693
6	#3	11′-6	6	#3	5′-0	5	#3	14′-6	6	#3	22′-0	1.44	.494
9	#3	11′-6	7	#3	5′-0	7	#3	14′-6	7	#3	22′-0	1.86	.515
6	#4	11′-6	9	#3	5′-0	9	#3	14′-6	9	#3	22′-0	2.36	.535
7	#4	11′-6	7	#4	5′-0	6	#4	14′-6	7	#4	22′-0	2.84	.535
8	#4	11′-6	7	#4	5′-0	7	#4	14′-6	7	#4	22′-0	3.10	.580
9	#4	11′-6	7	#4	5′-0	7	#4	14′-6	7	#4	22′-3	3.32	.625
7	#5	11′-6	9	#4	5′-0	8	#4	14′-6	9	#4	22′-3	3.92	.669
8	#5	11′-6	10	#4	5′-0	9	#4	14′-6	10	#4	22′-3	4.36	.714
7	#3	12′-0	6	#3	5′-3	6	#3	15′-6	6	#3	23′-3	1.50	.535
9	#3	12′-0	8	#3	5′-3	8	#3	15′-6	8	#3	23′-3	2.00	.535
7	#4	12′-0	6	#4	5′-3	6	#4	15′-6	6	#4	23′-3	2.60	.560
8	#4	12′-0	7	#4	5′-3	7	#4	15′-6	7	#4	23′-3	2.90	.580
9	#4	12′-0	8	#4	5′-3	8	#4	15′-6	8	#4	23′-3	3.22	.604
11	#4	12′-0	8	#4	5′-3	8	#4	15′-6	8	#4	23′-6	3.52	.648
8	#5	12′-0	6	#5	5′-3	6	#5	15′-6	6	#5	23′-6	4.13	.714
9	#5	12′-0	7	#5	5′-3	7	#5	15′-6	7	#5	23′-6	4.70	.755

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—
BANDS PERPENDICULAR TO AN EXTERIOR WALL

For Bands Parallel to Walls or Not in End Spans, Use "Two-Way Flat Slab Floors, Square Panels—Interior Panels," Pages 194 to 201.



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Superimposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant.	Trussed Bar No.	Trussed Length	Top at Ext. Col. Quant.	Top at Ext. Col. Bar No.	Top at Ext. Col. Length
	50	6	3	9	#3	16′-9	9	#3	24′-6	9	#3	6′-9
	100	6¼	3½	7	#4	16′-9	7	#4	24′-6	8	#4	6′-9
L = 19′-0	150	6½	3½	9	#4	16′-9	9	#4	24′-6	11	#4	6′-9
	200	7	3½	9	#4	16′-9	10	#4	24′-9	10	#4	6′-9
	250	7¼	3¾	10	#4	16′-9	11	#4	24′-9	12	#4	6′-9
D = 7′-3	300	7¾	4	7	#5	16′-9	8	#5	24′-9	9	#5	6′-9
	400	8½	4¼	8	#5	16′-9	9	#5	25′-0	9	#5	6′-9
C = 4′-0	500	9	4½	9	#5	16′-9	10	#5	25′-0	10	#5	6′-9
	50	6½	3¼	10	#3	17′-6	10	#3	26′-0	11	#3	7′-6
	100	6¾	3½	13	#3	17′-6	13	#3	26′-0	15	#3	7′-6
L = 20′-0	150	7	3½	9	#4	17′-6	9	#4	26′-0	10	#4	7′-6
	200	7	3½	11	#4	17′-6	11	#4	26′-0	12	#4	7′-6
D = 7′-6	250	7½	3¾	12	#4	17′-6	12	#4	26′-0	13	#4	7′-6
	300	8	4	13	#4	17′-6	13	#4	26′-0	15	#4	7′-6
C = 4′-6	400	9	4½	9	#5	17′-6	10	#5	26′-3	10	#5	7′-6
	500	10	5	10	#5	17′-6	10	#5	26′-3	12	#5	7′-6
	50	7	3½	11	#3	18′-6	11	#3	27′-3	12	#3	7′-9
	100	7	3½	8	#4	18′-6	9	#4	27′-3	9	#4	7′-9
L = 21′-0	150	7	3½	10	#4	18′-6	11	#4	27′-3	12	#4	7′-9
	200	7½	3¾	12	#4	18′-6	12	#4	27′-3	13	#4	7′-9
D = 7′-9	250	8	4	8	#5	18′-6	9	#5	27′-6	9	#5	7′-9
	300	8½	4¼	9	#5	18′-6	9	#5	27′-6	11	#5	7′-9
C = 4′-6	400	9½	4¾	10	#5	18′-6	10	#5	27′-9	12	#5	7′-9
	500	10½	5¼	11	#5	18′-6	11	#5	27′-9	13	#5	7′-9
	50	7	3½	12	#3	19′-3	13	#3	28′-9	13	#3	8′-0
	100	7	3½	10	#4	19′-3	10	#4	28′-9	11	#4	8′-0
L = 22′-0	150	7	3½	12	#4	19′-3	12	#4	28′-9	14	#4	8′-0
	200	7½	3¾	9	#5	19′-3	9	#5	28′-9	10	#5	8′-0
D = 8′-3	250	8	4	10	#5	19′-3	10	#5	28′-9	12	#5	8′-0
	300	8½	4¼	10	#5	19′-3	11	#5	28′-9	12	#5	8′-0
C = 5′-0	400	9½	4¾	12	#5	19′-3	12	#5	29′-0	13	#5	8′-0
	500	10½	5¼	13	#5	19′-3	13	#5	29′-0	15	#5	8′-0

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS— BANDS PERPENDICULAR TO AN EXTERIOR WALL

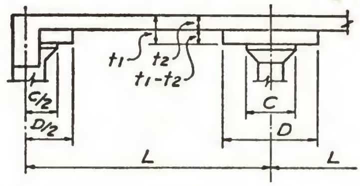
$f_s = 20{,}000$ psi
$f_c = 1{,}350$ psi
$v_c = 90$ psi
$u = 300$ psi

Each Column Strip			Each Middle Strip									Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor *
Top at Int. Col.			Top at Ext. Col.			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
9	#3	12′-9	7	#3	5′-6	7	#3	16′-3	7	#3	24′-6	1.67	.535
9	#4	12′-9	10	#3	5′-6	9	#3	16′-3	10	#3	24′-6	2.23	.562
9	#4	12′-9	7	#4	5′-6	7	#4	16′-3	7	#4	24′-6	2.91	.584
9	#4	12′-9	8	#4	5′-6	7	#4	16′-3	8	#4	24′-9	3.10	.625
12	#4	12′-9	9	#4	5′-6	8	#4	16′-3	9	#4	24′-9	3.50	.650
9	#5	12′-9	9	#4	5′-6	9	#4	16′-3	9	#4	24′-9	3.90	.693
9	#5	12′-9	7	#5	5′-6	7	#5	16′-3	7	#5	25′-0	4.50	.758
10	#5	12′-9	8	#5	5′-6	7	#5	16′-3	8	#5	25′-0	4.95	.875
9	#3	13′-6	8	#3	5′-9	8	#3	17′-0	8	#3	26′-0	1.79	.580
14	#3	13′-6	11	#3	5′-9	10	#3	17′-0	11	#3	26′-0	2.31	.605
9	#4	13′-6	13	#3	5′-9	12	#3	17′-0	13	#3	26′-0	2.82	.625
11	#4	13′-6	9	#4	5′-9	8	#4	17′-0	9	#4	26′-0	3.33	.625
13	#4	13′-6	10	#4	5′-9	9	#4	17′-0	10	#4	26′-0	3.75	.671
14	#4	13′-6	11	#4	5′-9	10	#4	17′-0	11	#4	26′-0	4.14	.713
9	#5	13′-6	8	#5	5′-9	7	#5	17′-0	8	#5	26′-3	4.60	.805
12	#5	13′-6	8	#5	5′-9	8	#5	17′-0	8	#5	26′-3	5.00	.885
10	#3	14′-0	9	#3	6′-0	8	#3	18′-3	9	#3	27′-0	1.90	.625
8	#4	14′-0	7	#4	6′-0	6	#4	18′-3	7	#4	27′-0	2.70	.625
10	#4	14′-0	9	#4	6′-0	8	#4	18′-3	9	#4	27′-0	3.15	.625
12	#4	14′-0	10	#4	6′-0	9	#4	18′-3	10	#4	27′-0	3.53	.670
8	#5	14′-0	11	#4	6′-0	10	#4	18′-3	11	#4	27′-3	3.95	.715
10	#5	14′-0	8	#5	6′-0	7	#5	18′-3	8	#5	27′-3	4.30	.760
11	#5	14′-0	8	#5	6′-0	8	#5	18′-3	8	#5	27′-6	4.80	.850
13	#5	14′-0	9	#5	6′-0	9	#5	18′-3	8	#5	27′-6	5.25	.931
13	#3	14′-9	10	#3	6′-3	10	#3	19′-0	10	#3	28′-6	2.03	.625
10	#4	14′-9	8	#4	6′-3	7	#4	19′-0	8	#4	28′-6	2.76	.625
13	#4	14′-9	10	#4	6′-3	9	#4	19′-0	10	#4	28′-6	3.45	.625
10	#5	14′-9	7	#5	6′-3	7	#5	19′-0	7	#5	28′-6	4.00	.670
11	#5	14′-9	8	#5	6′-3	8	#5	19′-0	8	#5	28′-6	4.53	.715
11	#5	14′-9	9	#5	6′-3	8	#5	19′-0	9	#5	28′-6	4.65	.760
13	#5	14′-9	10	#5	6′-3	9	#5	19′-0	10	#5	28′-9	5.30	.842
14	#5	14′-9	10	#5	6′-3	10	#5	19′-0	10	#5	28′-9	5.75	.931

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—BANDS PERPENDICULAR TO AN EXTERIOR WALL

For Bands Parallel to Walls or Not in End Spans, Use "Two-Way Flat Slab Floors, Square Panels—Interior Panels," Pages 194 to 201.



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Superimposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant.	Trussed Bar No.	Trussed Length	Top at Ext. Col. Quant.	Top at Ext. Col. Bar No.	Top at Ext. Col. Length
	50	7¼	3¾	14	#3	20′-0	14	#3	30′-0	16	#3	8′-3
L = 23′-0	100	7½	3¾	10	#4	20′-0	11	#4	30′-0	11	#4	8′-3
	150	7½	3¾	13	#4	20′-0	13	#4	30′-0	15	#4	8′-3
D = 8′-6	200	8	4	9	#5	20′-0	10	#5	30′-0	11	#5	8′-3
	250	8½	4¼	10	#5	20′-0	11	#5	30′-0	13	#5	8′-3
C = 5′-0	300	9	4½	8	#6	20′-3	9	#6	30′-0	11	#6	8′-3
	400	10	5	9	#6	20′-3	9	#6	30′-0	11	#6	8′-3
	500	12	5	9	#6	20′-3	9	#6	30′-3	12	#6	8′-3
	50	7½	3¾	9	#4	21′-0	9	#4	31′-0	10	#4	8′-9
L = 24′-0	100	7½	3¾	12	#4	21′-0	12	#4	31′-0	16	#4	8′-9
	150	8	4	9	#5	21′-0	9	#5	31′-0	11	#5	8′-9
	200	8½	4½	10	#5	21′-0	11	#5	31′-0	12	#5	8′-9
D = 8′-9	250	9	4½	8	#6	21′-3	8	#6	31′-0	10	#6	8′-9
	300	9½	4¾	9	#6	21′-3	9	#6	31′-0	10	#6	8′-9
C = 5′-6	400	11	4¾	9	#6	21′-3	10	#6	31′-6	12	#6	8′-9
	500	13	4¾	9	#6	21′-3	10	#6	31′-6	13	#6	8′-9
	50	7¾	4	10	#4	21′-6	10	#4	32′-6	13	#4	9′-0
L = 25′-0	100	8	4	13	#4	21′-6	13	#4	32′-6	14	#4	9′-0
	150	8½	4¼	10	#5	21′-6	10	#5	32′-6	12	#5	9′-0
D = 9′-3	200	9	4½	11	#5	21′-6	12	#5	32′-6	13	#5	9′-0
	250	9½	4¾	13	#5	21′-6	13	#5	32′-6	15	#5	9′-0
C = 5′-6	300	10	5	10	#6	21′-9	10	#6	32′-9	12	#6	9′-0
	400	11	5½	10	#6	21′-9	11	#6	32′-9	12	#6	9′-0
	500	13	5½	11	#6	21′-9	11	#6	33′-0	13	#6	9′-0
	50	8¼	4¼	11	#4	22′-6	11	#4	34′-0	13	#4	9′-3
L = 26′-0	100	8½	4¼	14	#4	22′-6	14	#4	34′-0	16	#4	9′-3
	150	8¾	4½	11	#5	22′-6	11	#5	34′-0	12	#5	9′-3
	200	9	4¾	12	#5	22′-6	13	#5	34′-0	14	#5	9′-3
D = 9′-6	250	9½	4¾	10	#6	22′-9	10	#6	34′-0	12	#6	9′-3
	300	10½	5	10	#6	22′-9	11	#6	34′-0	12	#6	9′-3
C = 5′-6	400	11½	5¾	12	#6	22′-9	12	#6	34′-0	14	#6	9′-3
	500	13½	6	12	#6	22′-9	13	#6	34′-3	14	#6	9′-3

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS—BANDS PERPENDICULAR TO AN EXTERIOR WALL

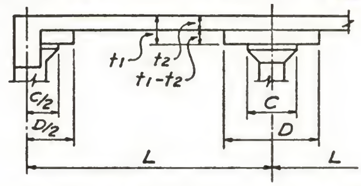
f_s = 20,000 psi
f_c = 1,350 psi
v_c = 90 psi
u = 300 psi

Each Column Strip			Each Middle Strip									Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor *
Top at Int. Col.			Top at Ext. Col.			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
15	#3	15′-6	11	#3	6′-6	11	#3	19′-9	11	#3	29′-6	2.13	.650
10	#4	15′-6	9	#4	6′-6	8	#4	19′-9	9	#4	29′-6	2.83	.670
13	#4	15′-6	11	#4	6′-6	10	#4	19′-9	11	#4	29′-6	3.53	.670
10	#5	15′-6	8	#5	6′-6	7	#5	19′-9	8	#5	29′-6	4.05	.714
11	#5	15′-6	9	#5	6′-6	8	#5	19′-9	9	#5	29′-6	4.50	.760
8	#6	15′-6	9	#5	6′-6	9	#5	19′-9	9	#5	29′-6	4.95	.803
10	#6	15′-6	10	#5	6′-6	10	#5	19′-9	10	#5	29′-6	5.54	.891
12	#6	15′-6	10	#5	6′-6	10	#5	19′-9	10	#5	29′-6	5.75	1.060
9	#4	16′-0	7	#4	6′-9	7	#4	20′-9	7	#4	30′-9	2.34	.670
12	#4	16′-0	10	#4	6′-9	9	#4	20′-9	10	#4	30′-9	3.11	.670
10	#5	16′-0	8	#5	6′-9	7	#5	20′-9	8	#5	30′-9	3.73	.714
12	#5	16′-0	9	#5	6′-9	8	#5	20′-9	9	#5	30′-9	4.27	.760
9	#6	16′-0	9	#5	6′-9	9	#5	20′-9	9	#5	30′-9	4.68	.803
10	#6	16′-0	10	#5	6′-9	10	#5	20′-9	10	#5	30′-9	5.20	.847
11	#6	16′-0	11	#5	6′-9	10	#5	20′-9	11	#5	31′-3	5.67	.972
14	#6	16′-0	11	#5	6′-9	11	#5	20′-9	11	#5	31′-3	6.00	1.139
12	#4	16′-9	8	#4	7′-0	8	#4	21′-3	8	#4	32′-3	2.55	.692
14	#4	16′-9	10	#4	7′-0	10	#4	21′-3	10	#4	32′-3	3.18	.715
11	#5	16′-9	12	#4	7′-0	12	#4	21′-3	12	#4	32′-3	3.85	.758
12	#5	16′-9	14	#4	7′-0	14	#4	21′-3	14	#4	32′-3	4.40	.803
13	#5	16′-9	16	#4	7′-0	15	#4	21′-3	16	#4	32′-3	4.93	.847
11	#6	16′-9	11	#5	7′-0	11	#5	21′-3	11	#5	32′-6	5.59	.892
12	#6	16′-9	12	#5	7′-0	12	#5	21′-3	12	#5	32′-6	5.98	.982
13	#6	16′-9	12	#5	7′-0	12	#5	21′-3	12	#5	32′-9	6.27	1.148
13	#4	17′-6	9	#4	7′-3	9	#4	22′-3	9	#4	33′-6	2.67	.714
14	#4	17′-6	11	#4	7′-3	11	#4	22′-3	11	#4	33′-6	3.33	.717
12	#5	17′-6	14	#4	7′-3	13	#4	22′-3	14	#4	33′-6	4.03	.760
13	#5	17′-6	16	#4	7′-3	15	#4	22′-3	16	#4	33′-6	4.64	.805
11	#6	17′-6	11	#5	7′-3	11	#5	22′-3	11	#5	33′-6	5.28	.848
13	#6	17′-6	13	#5	7′-3	13	#5	22′-3	14	#5	33′-6	5.70	.976
13	#6	17′-6	14	#5	7′-3	13	#5	22′-3	14	#5	33′-6	6.45	1.026
14	#6	17′-6	14	#5	7′-3	13	#5	22′-3	14	#5	34′-0	6.70	1.195

* These cubic foot quantities include drop panel but not column capital.

TWO-WAY FLAT SLAB FLOORS, SQUARE PANELS— BANDS PERPENDICULAR TO AN EXTERIOR WALL

For Bands Parallel to Walls or Not in End Spans, Use "Two-Way Flat Slab Floors, Square Panels—Interior Panels," Pages 194 to 201.



For general instructions and notes on the use of this table, see pages 189 to 193.

Span / Drop / Cap	Safe Superimposed Load (psf)	Slab Thickness t_2 (in.)	Drop Panel Thickness (t_1-t_2) (in.)	Each Column Strip: Straight Quant.	Straight Bar No.	Straight Length	Trussed Quant.	Trussed Bar No.	Trussed Length	Top at Ext. Col. Quant.	Top at Ext. Col. Bar No.	Top at Ext. Col. Length
L = 27′-0	50	8½	4¼	12	#4	23′-3	12	#4	35′-0	12	#4	9′-9
	100	9	4½	9	#5	23′-3	10	#5	35′-0	12	#5	9′-9
	150	9	4½	12	#5	23′-3	12	#5	35′-0	13	#5	9′-9
	200	9½	4¾	9	#6	23′-6	10	#6	35′-0	11	#6	9′-9
D = 10′-0	250	10	5	10	#6	23′-6	11	#6	35′-0	12	#6	9′-9
	300	10½	5¼	11	#6	23′-6	12	#6	35′-0	13	#6	9′-9
C = 6′-0	400	12	6	13	#6	23′-6	13	#6	35′-3	13	#6	9′-9
	500	13½	6	10	#7	24′-0	10	#7	35′-3	12	#7	9′-9
	50	8½	4¼	13	#4	24′-0	14	#4	36′-3	14	#4	10′-0
L = 28′-0	100	9	4½	11	#5	24′-0	11	#5	36′-3	12	#5	10′-0
	150	9	4¾	13	#5	24′-0	13	#5	36′-3	16	#5	10′-0
	200	9½	4¾	10	#6	24′-3	11	#6	36′-6	12	#6	10′-0
D = 10′-6	250	10½	5¼	12	#6	24′-3	12	#6	36′-6	12	#6	10′-0
	300	11½	5½	13	#6	24′-3	13	#6	36′-6	13	#6	10′-0
C = 6′-0	400	12½	6	11	#7	24′-9	11	#7	36′-9	11	#7	10′-0
	500	14	7	11	#7	24′-9	11	#7	36′-9	12	#7	10′-0
	50	8¾	4½	15	#4	25′-0	15	#4	37′-6	16	#4	10′-3
L = 29′-0	100	9¼	4¾	12	#5	25′-0	12	#5	37′-6	14	#5	10′-3
	150	9½	5	14	#5	25′-0	14	#5	37′-6	16	#5	10′-3
	200	10	5	11	#6	25′-3	12	#6	37′-6	15	#6	10′-3
D = 11′-0	250	11	5½	12	#6	25′-3	13	#6	37′-6	12	#6	10′-3
	300	11½	5½	10	#7	25′-9	11	#7	37′-9	10	#7	10′-3
C = 6′-6	400	13	6¼	11	#7	25′-9	12	#7	37′-9	11	#7	10′-3
	500	14½	7	12	#7	25′-9	13	#7	38′-3	12	#7	10′-3
	50	9	4½	10	#5	25′-9	11	#5	39′-0	11	#5	10′-9
L = 30′-0	100	9½	4¾	13	#5	25′-9	13	#5	39′-0	15	#5	10′-9
	150	10	5	11	#6	26′-0	11	#6	39′-3	12	#6	10′-9
D = 11′-3	200	10½	5¼	12	#6	26′-0	12	#6	39′-3	15	#6	10′-9
	250	11½	5½	13	#7	26′-0	14	#6	39′-3	16	#6	10′-9
	300	12	5¾	11	#7	26′-6	11	#7	39′-6	11	#7	10′-9
C = 6′-6	400	13½	6½	12	#7	26′-6	12	#7	39′-6	14	#7	10′-9
	500	15	7	12	#7	26′-6	13	#7	39′-9	16	#7	10′-9

TWO-WAY FLAT SLAB FLOORS; SQUARE PANELS—BANDS PERPENDICULAR TO AN EXTERIOR WALL

$f_s = 20{,}000$ psi
$f_c = 1{,}350$ psi
$v_c = 90$ psi
$u = 300$ psi

Each Column Strip			Each Middle Strip									Weight of Steel (psf)	Average Cubic Feet of Concrete Per Square Foot of Floor *
Top at Int. Col.			Top at Ext. Col.			Straight			Trussed				
Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length	Quant.	Bar No.	Length		
13	#4	18′-0	10	#4	7′-6	9	#4	23′-0	10	#4	34′-9	2.76	.759
10	#5	18′-0	12	#4	7′-6	12	#4	23′-0	12	#4	34′-9	3.49	.803
12	#5	18′-0	15	#4	7′-6	14	#4	23′-0	15	#4	34′-9	4.19	.803
10	#6	18′-0	11	#5	7′-6	11	#5	23′-0	11	#5	34′-9	4.95	.846
11	#6	18′-0	12	#5	7′-6	12	#5	23′-0	12	#5	34′-9	5.46	.892
12	#6	18′-0	13	#5	7′-6	13	#5	23′-0	13	#5	34′-9	5.90	.937
15	#6	18′-0	11	#6	7′-6	11	#6	23′-0	12	#6	35′-0	6.75	1.072
13	#7	18′-0	11	#6	7′-6	11	#6	23′-0	11	#6	35′-0	7.25	1.195
13	#4	18′-9	11	#4	7′-9	10	#4	23′-9	11	#4	36′-0	2.97	.758
11	#5	18′-9	14	#4	7′-9	13	#4	23′-9	14	#4	36′-0	3.73	.802
15	#5	18′-9	17	#4	7′-9	16	#4	23′-9	17	#4	36′-0	4.57	.806
11	#6	18′-9	12	#5	7′-9	12	#5	23′-9	12	#5	36′-3	5.24	.851
17	#6	18′-9	15	#5	7′-9	15	#5	23′-9	15	#5	36′-3	5.80	.980
17	#6	18′-9	15	#5	7′-9	15	#5	23′-9	16	#5	36′-3	6.40	1.064
13	#7	18′-9	12	#6	7′-9	12	#6	23′-9	13	#6	36′-3	7.52	1.153
13	#7	18′-9	12	#6	7′-9	12	#6	23′-9	13	#6	36′-3	7.55	1.246
16	#4	19′-6	12	#4	8′-0	11	#4	24′-9	12	#4	37′-3	3.15	.781
13	#5	19′-6	15	#4	8′-0	14	#4	24′-9	15	#4	37′-3	3.92	.805
20	#5	19′-6	18	#4	8′-0	17	#4	24′-9	18	#4	37′-3	4.66	.850
12	#6	19′-6	13	#5	8′-0	13	#5	24′-9	13	#5	37′-3	5.43	.893
16	#6	19′-6	15	#5	8′-0	15	#5	24′-9	16	#5	37′-3	6.05	.981
11	#7	19′-6	17	#5	8′-0	17	#5	24′-9	17	#5	37′-6	6.60	1.064
15	#7	19′-6	13	#6	8′-0	13	#6	24′-9	14	#6	37′-6	7.60	1.198
15	#7	19′-6	13	#6	8′-0	13	#6	24′-9	14	#6	38′-0	7.90	1.288
10	#5	20′-6	13	#4	8′-6	12	#4	25′-6	13	#4	38′-6	3.29	.803
13	#5	20′-6	16	#4	8′-6	15	#4	25′-6	16	#4	38′-6	4.07	.846
11	#6	20′-6	12	#5	8′-6	12	#5	25′-6	12	#5	38′-9	4.95	.892
14	#6	20′-6	14	#5	8′-6	13	#5	25′-6	14	#5	38′-9	5.70	.937
15	#6	20′-6	15	#5	8′-6	15	#5	25′-6	15	#5	38′-9	6.30	1.025
14	#7	20′-6	12	#6	8′-6	12	#6	25′-6	14	#6	39′-0	7.00	1.062
14	#7	20′-6	13	#6	8′-6	13	#6	25′-6	14	#6	39′-0	7.70	1.141
16	#7	20′-6	14	#6	8′-6	14	#6	25′-6	15	#6	39′-0	8.35	1.330

* These cubic foot quantities include drop panel but not column capital.

REINFORCED CONCRETE BEAMS
(WORKING LOAD METHOD)

SINGLE SPAN SIMPLY SUPPORTED

The first set of tables, page 214, gives the total safe uniform load per lineal foot (live and dead) * on rectangular and tee beams for single spans simply supported at each end, computed in conformity with the American Concrete Institute's "Building Code Requirements for Reinforced Concrete (ACI 318-56)"; for span lengths varying from 10 to 30 feet in two-foot multiples; for one set of stresses, viz. $f_s = 20{,}000$ psi and $f_c = 1350$ psi; for depths from 12 to 30 inches, in two-inch multiples; for three stem widths (b') in each depth; with two choices of bar combination for each stem width, the first being that which produces balanced reinforcement ($p = 0.0136$) for a rectangular beam of the stem size given (b'), and the second about double that amount, requiring a rectangular beam of twice the width first given, or what is the same thing so far as calculations are concerned, the addition of a flange with width (b) twice that of the stem ($b = 2b'$). The depth of flange equals the depth to the neutral axis as fixed by the chosen stresses. In using these tables the tributary slab will practically always be thinner than the depth in the table. Approximate results satisfactory for many purposes will be given by widening the flange (b) so that the total area outside of the beam stem is the same as that given by the flange scheduled in the tables. This is slightly on the safe side and the tee may be computed accurately, if desired, always keeping within the limitations of ACI 705.

The necessary stirrups are given in the tables. For each safe load given, there is also scheduled a mark for a stirrup combination. The make-up of these combinations is tabulated on page 230. The first digit in the mark signifies the size of bar to use, the second (and possible third) digit gives the number of stirrups required in each end of the beam, and the final letter designates the required group spacing as given in the table. Since each variation of load, span or beam size requires a different stirrup arrangement, the combinations given, while on the safe side, are not always the most economical. The example on page 212 shows how a more exact arrangement can be determined. After designing stirrups as illustrated on pages 86-87 two stirrups were added at $d/2$ to comply with ACI 318-56 801(d).

Bond has been computed on the basis of deformed bars meeting ASTM A305. Plain round bars or deformed bars not meeting ASTM A305 will not afford sufficient bond to develop the values given in the tables.

The safe carrying capacity in these beam tables was obtained by taking the least of the values determined by shear, bond or flexure.

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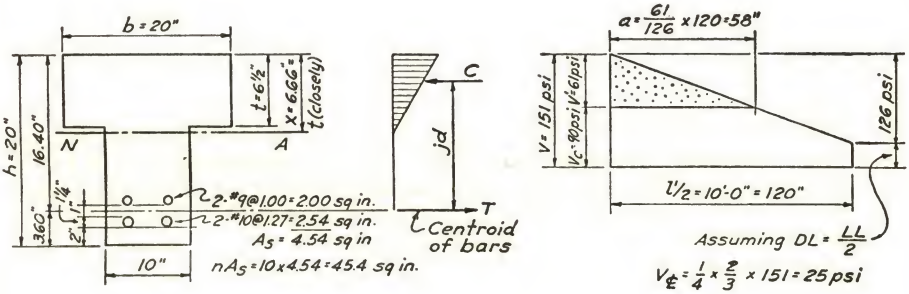
* In the various slab tables throughout this book, the weight of the slab has been deducted so that the values given in the tables are the safe superimposed loads. In the case of beams, there is no advantage in deducting the minor weight of the beam stem, as it is the weight of the tributary slab that is the main element of dead load. So in these tables the capacity given is the total safe load, dead plus live.

REINFORCED CONCRETE BEAMS
SINGLE SPAN SIMPLY SUPPORTED

One of the main advantages of reinforced concrete construction is the wide range of sizes open to the designer. Concrete beams may be wide and shallow, narrow and deep, or of more economical proportions with d equal to about two to three times b'. With deep beams, relatively little reinforcement is needed (though p must $\gtrless$ 0.005 $b'd$; ACI 702*e1*); while with shallow beams, a wide tee and considerably heavier reinforcement are required. Sometimes compressive steel is used when space is not available for a wide tee. It is impracticable to tabulate all possible beam sizes and steel combinations, so the following example may prove useful for those who wish to design beyond the scope of the tables or to see how they were prepared:—

Example—For the table on pages 214-215, determine the safe carrying capacity on a span of 20 feet of a 10 x 20 in. beam stem reinforced with 2-#10 bars in the bottom and 2-#9 bars trussed, placed in two layers; check the flange value of 6½ x 20 in., and also the stirrup combination 310*f*.



Solution—Safe load tables reverse the process of design by selecting a section and determining its safe capacity. Take this as a tee beam and assume that the neutral axis is at the bottom of the flange. The centroid of the transformed section can be located thus*:—

$$\frac{20x^2}{2} - 45.4(16.40 - x) = 0$$

$$10x^2 + 45.4x = 745$$

$$x^2 + 4.54x = 74.5$$

$$x^2 + 4.54x + (2.27)^2 = 74.5 + 5.2 = 79.7 \qquad x = 6.66 \text{ in.} = 0.40d \text{ (closely)}$$

Hence, if the tee were 6.66 in. deep, it would extend all the way down to the neutral axis, making this in effect a rectangular beam. The difference between 6.5 and 6.66 is too slight to make any appreciable difference in the computation.

Flexure—The arm of the internal couple $= jd = 16.40 - \dfrac{6.66}{3} = 14.18$ in.

$$M_s = A_s f_s jd = 4.54 \times 20{,}000 \times 14.18 = 1{,}289{,}000 \text{ lb-in.} = \frac{wl'^2 12}{8}$$

$$M_c = bkd\frac{f_c}{2}jd = 20 \times 6.66 \times \frac{1350}{2} \times 14.18 = 1{,}277{,}000 \text{ lb-in.}$$

This is essentially a beam with "balanced reinforcement."

* Computations in tables are carried to a greater degree of precision than is significant or customary in design offices.

REINFORCED CONCRETE BEAMS
SINGLE SPAN SIMPLY SUPPORTED

$$w = \frac{M}{l'^2\frac{12}{8}} = \frac{1{,}277{,}000\,^* \times 2}{3 \times 20 \times 20} = \mathbf{2130} \text{ plf (in table)}$$

Bond—On 2-#10 bottom bars:—

$$V = \Sigma ojdu = 2 \times 3.99 \times 14.18 \times 300 = 34{,}000 \text{ lb} = \frac{wl'}{2}$$

$$w = \frac{34{,}000}{10} = 3400 \text{ plf, or more than allowed by flexure}$$

Shear—For spacing stirrups, it is not practicable to work backwards, so take the capacity from flexure, 2130 plf, and design the web reinforcement to take care of this:—

$$v = \frac{V}{bjd} = \frac{2130 \times 10}{10 \times 14.18} = 151 < 360 \text{ psi allowable}$$

From the figure on page 211, allowing $v_c = 90$ psi on the concrete leaves 61 psi on the reinforcement, and the distance to be covered is easily computed:—

$$a = \frac{v_s}{\Delta v}\frac{l'}{2} = \frac{61}{126} \times 120 = 58 \text{ in.}$$

Then the total area of web reinforcement in one end of the beam can be computed:—

$$A_v = \frac{bv_sa}{2f_v} = \frac{10 \times 61 \times 58}{2 \times 20{,}000} = 0.88 \text{ sq in.}$$

Minimum requirement in distance a—4-#3 U-stirrups = 0.88 sq in.
The spacing figured by the method explained on pages 86-91 is 4, 8 (10) (15), since the spacing must not exceed $d/2$ (ACI 806a), use 5-#3 stirrups spaced 4, 8, 8, 8, 8. Moreover, the stirrups must cover a distance, ACI 801 (d), a + d = 58 + 16 = 74 in. spaced at not over $d/2$, or 9-#3 U-shaped stirrups spaced 4, 8 @ 8. The combination furnished in the table is:—10f = 10-#3 U-shaped stirrups spaced 2, 5, 7, 7 @ 8. This is more than adequate for this particular load and span, but an attempt was made in the table to keep a reasonable number of practicable stirrup combinations.

Had this been a continuous or restrained beam or frame without a monolithic slab to provide T-beam action, additional web reinforcement would have had to be provided as per ACI 801e.

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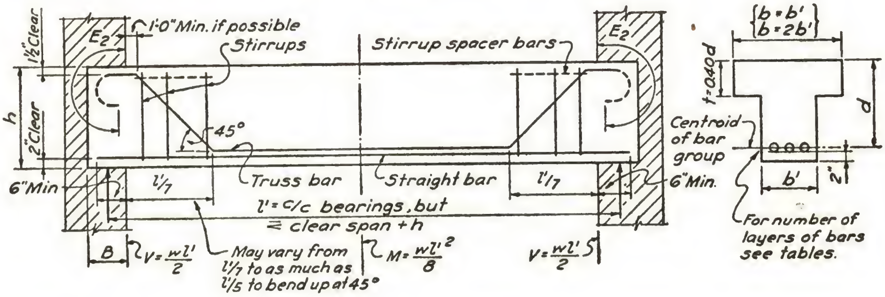
* If the beam is a freestanding tee beam, then M_c is the value to use. If the beam is monolithic with a floor slab, the width of tee, b, is arbitrary and a slightly greater width might be taken, increasing jd to $16.40 - \frac{6.50}{3} = 14.23$ in., and M_s to 1,292,000 lb-in., which is the determining value.

REINFORCED CONCRETE BEAMS
SINGLE SPAN SIMPLY SUPPORTED

Applies to the tables on pages 214-217.



STRESSES:—

f_s = 20,000 psi
f'_c = 3000 psi
f_c = 1350 psi
v_c = 90 psi

v = 360 psi
u = 300 psi for bottom bars
= 210 psi for top bars with over 12" of concrete under them

CODES:—

"Building Code Requirements for Reinforced Concrete (ACI 318-56)," also "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

E_1 = 6 in. minimum for bottom bars
E_2 = 17 bar diameters (24 diameters if $d > 12$ in.) (straight if possible, bent if necessary).
B = ordinary 8" minimum and sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall is made (see page 99).

For stirrup combinations, see page 230.
Values in the tables are given as follows:—

Given Section hb' { 1st horizontal line:— $b = b'$ = Rectangular Beam
2nd horizontal line:— $b = 2b'$ = Tee Beam

REINFORCED CONCRETE BEAMS—SINGLE SPAN

For limitations and explanation of use of this table, see pages 210-213

	Stem		Flange		Bar Combinations					TOTAL SAFE CARRYING CAPACITY* (plf)—UNIFORMLY DISTRIBUTED, Span l' in Feet							
					Straight		Trussed			10		12		14		16	
	h (in.)	b′ (in.)	t = 0.4d (in.)	b (in.)	No.	Size	No.	Size	No. Layers	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load
1	12	6	—	6	2	#4	2	#4	2	—	784	—	544	—	399	—	306
2		8	—	8	2	#5	1	#6	1	—	1201	—	835	—	614	—	470
3			3½	16	2	#7	2	#6	2	210a	1990	29a	1380	28a	1016	26a	778
4		10	—	10	2	#6	1	#6	1	—	1493	—	1038	—	761	—	583
5			3½	20	3	#6	2	#7	2	212a	2544	210a	1768	28a	1298	26a	994
6	14	8	—	8	2	#6	1	#6	1	24b	1734	—	1200	—	885	—	678
7			4½	16	2	#8	1	#8	2	212b	2834	210b	1968	29b	1444	27a	1108
8		10	—	10	2	#7	1	#6	1	24b	2190	—	1520	—	1119	—	855
9			4½	20	2	#9	1	#9	1	310b	3543	211a	2460	210b	1810	27a	1385
10		12	—	12	2	#7	2	#6	1	24b	2580	—	1791	—	1318	—	1010
11			4½	24	2	#10	1	#10	1	312b	4803	312c	3500	311b	2570	212c	1970
12	16	8	—	8	2	#6	2	#5	2	25b	2061	23c	1430	—	1050	—	805
13			5	16	2	#8	2	#7	2	310c	4009	310d	2780	210d	2042	210d	1565
14		10	—	10	3	#5	3	#5	2	25b	2654	23c	1840	—	1350	—	1034
15			5	20	3	#8	2	#7	2	312d	5147	310d	3570	212d	2620	210d	2020
16		12	—	12	3	#6	2	#6	1	26c	3493	25b	2425	23c	1780	—	1364
17			5	24	3	#9	2	#7	2	313a	6004	312d	4165	310d	3060	212d	2342
18	18	8	—	8	3	#5	{1 2	#5} #4}	2	26d	2827	25c	1960	24c	1440	—	1103
19			6	16	2	#9	2	#7	2	313b	5371	311c	3730	311c	2740	212f	2095
20		10	—	10	2	#8	1	#7	1	26d	3832	25c	2660	24c	1955	—	1498
21			6	20	4	#7	2	#8	2	313b	6714	312f	4660	311c	3420	310e	2620
22		12	—	12	2	#8	1	#9	1	26d	4598	25c	3190	24c	2340	—	1795
23			6	24	3	#8	3	#8	2	312e	7153	313b	5600	312f	4110	311c	3150
24	20	8	—	8	2	#7	2	#5	2	26f	3546	26d	2465	25c	1810	23d	1388
25			6½	16	2	#9	2	#8	2	49g	6067	410f	4748	410f	3485	311e	2670
26		10	—	10	2	#8	1	#8	1	36f	4876	26f	3385	26d	2486	25d	1902
27			6½	20	2	#10	2	#9	2	312e	6856	313c	5700	313d	4360	312g	3340
28		12	—	12	2	#9	2	#6	1	36f	5872	36f	4075	25d	2995	25d	2290
29			6½	24	4	#8	3	#8	2	411d	10413	411d	7230	410f	5310	312g	4070
30	22	8	—	8	2	#7	2	#6	2	36f	4441	36f	3080	26f	2265	25d	1735
31			7½	16	3	#9	1	#9	2	410g	8601	410i	5975	410i	4390	311g	3360
32		10	—	10	3	#6	3	#6	2	36f	5593	36f	3880	26f	2850	25d	2180
33			7½	20	3	#10	1	#10	2	411f	10601	410h	7370	410i	5400	312h	4140
34		12	—	12	2	#8	2	#8	1	36f	6400	36f	5040	26f	3700	26d	2840
35			7½	24	3	#10	3	#8	2	411f	11531	412g	8970	412h	6590	410i	5050

* Deduct dead load from these values.

Where dash occurs in stirrup column, no stirrups are required.

REINFORCED CONCRETE BEAMS—SINGLE SPAN

TOTAL SAFE CARRYING CAPACITY * (plf)— UNIFORMLY DISTRIBUTED

Span l' in Feet

18		20		22		24		26		28		30		
Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	
—	242													1
—	371													2
—	614													3
—	460													4
23a	785													5
—	535	—	432											6
26b	875	23b	709											7
—	675	—	547											8
26b	1093	23b	886											9
—	795	—	645											10
210b	1558	28b	1260	26b	1040	23b	873							11
—	636	—	515	—	426	—	358							12
28d	1236	27b	1002	25b	829	23c	695							13
—	817	—	663	—	548	—	460							14
29c	1586	27b	1285	26c	1061	24b	891							15
—	1079	—	873	—	722	—	606							16
28d	1852	27b	1500	25b	1240	23c	1041							17
—	871	—	707	—	584	—	490	—	418					18
211c	1660	29e	1342	28e	1110	27d	932	25c	795	24c	685			19
—	1180	—	958	—	792	—	665	—	567	—	—			20
211c	2070	29e	1680	28e	1390	27d	1165	25c	974	23c	858			21
—	1419	—	1150	—	950	—	798	—	678	—	—			22
213b	2488	211c	2019	29e	1668	27d	1400	26d	1192	24c	1030			23
—	1096	—	888	—	734	—	616	—	525	—	452	—	394	24
310f	2110	310f	1710	29g	1411	28f	1185	27f	1010	26f	871	25d	760	25
—	1503	—	1220	—	1008	—	845	—	721	—	622	—	542	26
311e	2640	310f	2130	211d	1770	29g	1485	27f	1265	26f	1092	25d	951	27
—	1810	—	1470	—	1214	—	1019	—	869	—	750	—	652	28
311e	3218	310f	2600	211d	2150	29g	1810	27f	1540	26f	1330	25d	1158	29
23e	1370	—	1110	—	918	—	770	—	657	—	567	—	493	30
310i	2650	310i	2150	212h	1780	210i	1490	29j	1274	27h	1098	26g	957	31
23e	1723	—	1396	—	1153	—	970	—	827	—	713	—	620	32
311g	3270	310i	2650	310i	2190	210h	1840	29j	1570	27h	1352	26g	1180	33
25c	2240	—	1815	—	1500	—	1260	—	1075	—	928	—	808	34
313e	3985	313e	3230	310i	2670	310i	2240	210h	1910	29j	1645	27h	1435	35

$d = h - (2'' +$ distance from bottom of bars to their centroid).

REINFORCED CONCRETE BEAMS—SINGLE SPAN

For limitations and explanation of use of this table, see pages 210-213.

	Stem		Flange		Bar Combinations					TOTAL SAFE CARRYING CAPACITY* (plf)—UNIFORMLY DISTRIBUTED							
					Straight		Trussed			Span l' in Feet: 10		12		14		16	
	h (in.)	b′ (in.)	t = 0.4d (in.)	b (in.)	No.	Size	No.	Size	No. Layers	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load
1	24	10	—	10	3	#7	3	#5	2	37*i*	6665	36*h*	4630	26*h*	3400	25*f*	2602
2			8½	20	3	#9	{1, 2}	{#9, #8}	2	411*i*	11308	412*k*	9175	411*j*	6740	411*k*	5160
3		12	—	12	2	#10	1	#9	1	39*k*	8761	38*i*	6085	36*h*	4470	26*h*	3420
4			8½	24	3	#10	3	#9	2	411*i*	12696	413*f*	10580	412*k*	7998	412*l*	6110
5		14	—	14	2	#10	2	#8	1	38*i*	8967	38*i*	7120	37*i*	5225	26*h*	4000
6			8½	28	3	#11	2	#11	2	412*j*	13978	413*f*	11650	413*g*	9200	412*l*	7050
7	26	10	—	10	3	#7	3	#6	2	39*k*	8222	37*j*	5700	29*k*	4190	27*i*	3210
8			9	20	2	#11	2	#11	2	49*m*	10239	410*p*	8520	411*m*	7300	412*l*	6076
9		12	—	12	2	#10	1	#10	1	310*o*	9790	39*k*	7250	37*j*	5330	28*i*	4080
10			9	24	3	#10	{2, 1}	{#10, #9}	2	411*l*	13949	413*i*	11600	414*c*	9630	413*j*	7380
11		14	—	14	3	#9	2	#8	1	48*k*	12268	310*o*	8575	38*k*	6250	29*k*	4785
12			9	28	3	#11	3	#10	2	412*j*	15359	413*i*	12800	415*b*	10950	414*c*	8500
13	28	12	—	12	4	#7	4	#6	2			39*o*	8000	38*k*	5890	210*o*	4500
14			9½	24	6	#8	3	#9	3			417*a*	14490	415*b*	10630	414*d*	8130
15		14	—	14	4	#8	2	#8	2			310*o*	9252	38*k*	6800	37*j*	5200
16			9½	28	4	#11	2	#11	2			418*a*	17690	417*b*	13200	416*d*	10100
17		16	—	16	2	#11	2	#10	1			49*o*	11250	310*r*	8275	38*k*	6330
18			9½	32	5	#10	3	#10	2			419*a*	19850	418*a*	14580	416*d*	11180
19	30	12	—	12	3	#9	2	#8	2					39*o*	6820	37*n*	5215
20			10½	24	4	#11	2	#10	2					417*b*	13250	416*d*	10150
21		14	—	14	4	#8	2	#9	2					310*r*	7875	39*o*	6030
22			10½	28	6	#9	3	#10	3					419*b*	14600	418*b*	11190
23		16	—	16	4	#8	6	#6	2					47*n*	9100	39*o*	6960
24			10½	32	8	#9	4	#8	3					421*a*	16520	419*c*	12670

* Deduct dead load from these values.
Where dash occurs in stirrup column, no stirrups are required.

REINFORCED CONCRETE BEAMS—SINGLE SPAN

TOTAL SAFE CARRYING CAPACITY * (plf)— UNIFORMLY DISTRIBUTED

Span l' in Feet

18		20		22		24		26		28		30		
Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	
24*f*	2060	23*e*	1666	—	1380	—	1159	—	986	—	851	—	741	1
313*g*	4075	312*l*	3300	311*k*	2730	310*m*	2292	213*g*	1952	211*j*	1685	29*l*	1469	2
25*f*	2702	24*f*	2190	—	1810	—	1520	—	1296	—	1119	—	964	3
411*k*	4830	410*m*	3910	312*l*	3238	311*k*	2720	310*m*	2318	212*k*	2000	210*l*	1740	4
25*f*	3160	24*f*	2560	—	2118	—	1780	—	1515	—	1308	—	1140	5
412*l*	5560	411*k*	4510	410*m*	3730	312*l*	3135	310*m*	2670	213*f*	2300	210*l*	2000	6
26*h*	2518	25*f*	2055	24*f*	1698	—	1430	—	1215	—	1048	—	913	7
411*n*	4800	314*d*	3885	313*k*	3220	312*o*	2700	311*n*	2300	214*d*	1985	212*l*	1726	8
26*h*	3220	25*f*	2610	24*f*	2160	—	1815	—	1545	—	1332	—	1162	9
412*o*	5830	411*n*	4720	314*d*	3900	313*k*	3280	312*o*	2795	310*q*	2415	214*c*	2100	10
26*h*	3780	25*f*	3067	24*f*	2535	—	2130	—	1812	—	1564	—	1361	11
413*j*	6700	412*o*	5440	411*n*	4500	410*q*	3780	313*k*	3220	311*n*	2770	310*q*	2420	12
27*i*	3560	26*h*	2885	25*f*	2350	24*f*	2000	—	1705	—	1472	—	1282	13
413*k*	6420	412*o*	5200	316*e*	4300	314*g*	3620	313*k*	3080	312*o*	2660	310*q*	2320	14
27*i*	4110	26*h*	3338	25*f*	2758	24*f*	2312	—	1970	—	1700	—	1480	15
414*g*	7975	413*m*	6460	318*b*	5340	316*e*	4490	314*g*	3820	313*k*	3300	311*n*	2870	16
29*k*	5000	26*h*	4050	25*f*	3350	24*f*	2815	—	2400	—	2070	—	1800	17
414*g*	8820	413*m*	7150	413*k*	5910	316*e*	4960	314*g*	4220	313*k*	3650	311*n*	3170	18
29*o*	4125	27*l*	3340	26*h*	2760	25*f*	2320	24*f*	1980	—	1708	—	1480	19
414*g*	8020	413*p*	6490	412*s*	5370	317*c*	4510	315*e*	3840	313*p*	3318	312*s*	2885	20
29*o*	4760	27*l*	3860	26*h*	3190	25*f*	2680	24*f*	2280	—	1970	—	1715	21
416*e*	8840	415*e*	7150	414*h*	5920	413*p*	4970	318*c*	4240	315*e*	3650	314*h*	3180	22
37*n*	5500	29*o*	4450	26*h*	3680	25*f*	3090	24*f*	2640	—	2275	—	1980	23
418*b*	10000	416*e*	8100	414*h*	6700	414*h*	5620	413*p*	4790	316*e*	4130	314*h*	3599	24

$d = h - (2'' + \text{distance from bottom of bars to their centroid})$.

REINFORCED CONCRETE BEAMS
(WORKING LOAD METHOD)
CONTINUOUS END SPAN

The table on page 222 gives the total safe uniform load per lineal foot (live and dead) * on reinforced concrete beams for the end span only of continuous runs of beams, computed in conformity with the American Concrete Institute's "Building Code Requirements for Reinforced Concrete (ACI 318-56)"; for span lengths varying from 10 to 30 feet in two-foot multiples; for one set of stresses, viz. $f_s = 20{,}000$ psi and $f_c = 1350$ psi; for depths from 12 to 30 inches in two-inch multiples; for three widths (b', page 221) in each depth; with bar combinations which for a rectangular beam of the size given produce balanced reinforcement ($p = 0.0136$).

There is no great advantage in listing much more heavily reinforced continuous beams, because negative bending quite sharply limits their maximum capacity. This may be seen in a general fashion by considering a continuous rectangular beam of about the width and depth covered by these tables. Consider the negative moment at the first interior support when the length of the interior span equals 1.2 times that of the end span, and the bending moment equals 1/10 of w times the square of the average span length (ACI 701c). Taking as a practical limit a steel ratio of perhaps 2 or 2½ per cent, the value of $R = \dfrac{M}{bd^2}$ is around 450, and the (stem) width will then be $b' = \dfrac{M}{Rd^2} = \dfrac{w\,(1.1\,L)^2}{10 \times 450d^2} = \dfrac{wL^2}{3710d^2}$.

Turning to the positive moment in the end span, taking a bending moment of $wL^2/11$ (outer end freely supported without restraint), and computing the (flange) width (which is the same as the width of the tee beam in the previous tables, with superfluous concrete below the neutral axis removed), and taking $R = M/bd^2 = 235$ for balanced reinforcement, the flange width $b = \dfrac{M}{Rd^2} = \dfrac{wL^2}{11 \times 235d^2} = \dfrac{wL^2}{2585d^2}$. So for equal resistance to positive and negative moment $b = \dfrac{3710}{2585}\,b' = 1.43b'$. Thus the maximum useable flange width in this special case is 1.43 times the stem width and the maximum practical reinforcement is 1.43 times that required for balanced reinforcement of the stem. Beyond that limit, there is no advantage in increasing reinforcement or flange width.

*** In the various slab tables throughout this book, the weight of the slab has been deducted so that the values given in the tables are the safe superimposed loads. In the case of beams, there is no advantage in deducting the minor weight of the beam stem, as it is the weight of the tributary slab that is the main element of dead load. So in these tables the capacity given is the total safe load, dead plus live.**

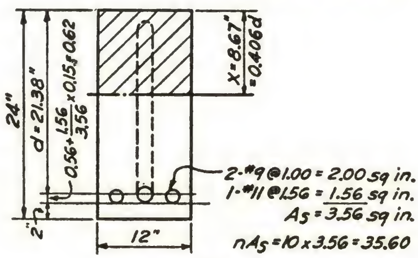
REINFORCED CONCRETE BEAMS
CONTINUOUS END SPAN

The limitations and arrangement of the following table parallel so closely those of the single span that pages 210 to 213 should be studied carefully before reading further. This applies especially to stirrup arrangements and bond, and to the fact that bars must meet ASTM A305, and that carrying capacity is taken as the least of the limits set by shear, bond, positive or negative flexure.

The 1956 ACI Building Code recommends a value for positive moment in end spans of $wl'^2/11$ where the outer end is freely supported (and that is used here) or of $wl'^2/14$ where the outer end is monolithic with a reinforced concrete frame (which will usually increase the capacity over what is given here).

The following example may prove useful for those who wish to design beyond the scope of the tables or to see how they were prepared:—

Example—For the table on pages 222-223, determine the safe carrying capacity on an end span of 20 feet of a 12 x 24 in. beam stem reinforced with 2-#9 bottom bars, 1-#11 truss bar, all in one layer, with 1-#9 added top bar over the support at the continuous end, assuming 1-#11 truss bar bent up and carried through from the adjacent span. Check the stirrup combination 39*l*.



Solution—From the figure determine first the distance "x" down to the neutral axis by equating statical moments of the transformed areas about the neutral axis:—

$$p = \frac{3.56}{12 \times 21.38} = 0.01387 > 0.0136$$

Slightly over balanced reinforcement

$$\frac{12x^2}{2} = 35.6\,(21.38 - x)$$

$$6x^2 + 35.6x = 761.2$$

$$x^2 + 5.93x + (2.97)^2 = 126.86 + 8.82 = 135.68$$

$$x = -2.97 \pm 11.64 = 8.67 \text{ in.} (0.406d)$$

Positive Flexure—The arm of the internal couple, jd, $= 21.38 - \frac{8.67}{3} = 18.5$ in.

$$M_s = A_s f_s jd = 3.56 \times 20{,}000 \times 18.5 = 1{,}317{,}000 \text{ lb-in.}$$

$$M_c = bkd\frac{f_c}{2}jd = 12 \times 8.67 \times \frac{1350}{2} \times 18.5 = 1{,}300{,}000$$

practically "balanced reinforcement."

Since the amount of tension steel is accurately established, while the amount of compression concrete can usually be increased by assuming a slightly wider strip of slab in selecting b, use $M_s = 1{,}317{,}000$ lb-in. and compute the safe load:—

$$w = \frac{M}{l'^2\frac{12}{11}} = \frac{1{,}317{,}000}{20 \times 20 \times \frac{12}{11}} = 3020 \text{ plf}$$

Bond on Bottom Bars—The bond on the bottom bars is to be computed in that part of the positive bending zone which has the highest external shear, i.e., either the free end or the point of inflection of the continuous end. Since the ACI Building Code sets the shear at the free end at $wl'/2$ and it is unlikely that the shear at the point of inflection will normally be that large:—

$$V = \Sigma ojdu = 2 \times 3.544 \times 18.5 \times 300 = 39{,}400 \text{ lb} = wl'/2$$

$$w = \frac{39{,}400}{10} = 3940 \text{ plf, or more than allowed by flexure.}$$

Shear—To allow for the fact that continuity in an end span increases the shear at the continuous end, ACI 701c requires an increase of 15 per cent over $wl'/2$. For spacing stirrups, it is not practicable to work backwards. Take the capacity of the beam at

REINFORCED CONCRETE BEAMS
CONTINUOUS END SPAN

3020 plf as computed for positive bending and design the web reinforcement to take care of this:—

$$V = 1.15\frac{wl'}{2} = 1.15 \times 3020 \times 10 = 34{,}700 \text{ lb}$$

$$v = \frac{V}{bjd} = \frac{34{,}700}{12 \times 18.5} = 156 \text{ psi} > 90 < 360 \text{ psi}$$

Deducting $v_c = 90$ psi leaves 66 psi to be carried by stirrups. Although there is a bent bar which might displace a stirrup or two, the sloping portion frequently is not well located for that purpose (being bent for positive and negative bending); also, such refinement is mainly used in large girders where making detailed layouts justifies the effort. Here stirrups are standardized in groups large enough to be on the safe side but only occasionally the absolute minimum.

The distance to be covered by stirrups is found by dividing $v_s = 66$ psi by the change in shear per lineal inch of span, $z = \frac{w}{bjd\,(12)} = \frac{3020}{12 \times 18.5 \times 12} = 1.135$ psi/in., then

$$a = \frac{66}{1.135} = 58 \text{ in.}$$

The total area of stirrups:—

$$A_v = \frac{bav_s}{2f_v} = \frac{12 \times 58 \times 66}{2 \times 20{,}000} = 1.15 \text{ sq in.}$$

Minimum requirement is 6-#3 U-stirrups = 1.32 sq in.

The spacing is figured by the method explained on pages 86-91 as 2, 6, 6, 7, 8, (12); but this last space exceeds $d/2$ (ACI 806a), and (ACI 801d) stirrups must cover a distance $(a + d)$, spaced at not over $d/2$, so use 8-#3 U-stirrups at 2, 6, 6, 7, 8, 9, 10, 10.

Had this been a continuous or restrained beam or frame without a monolithic slab to provide T-beam action, additional web reinforcement would have had to be provided as per ACI 801e. The combination furnished in the table is $39l$, or #3 stirrups, 9 at each end of the span, spaced 2, 5, 6, 7, 9, 10, 10, 10, 10, which is about as close as can be obtained with standardized groups.

The same arrangement will be used on the free end for simplicity in detailing and to prevent reversal in the field.

Negative Flexure—By Code, the negative moment factor is to be taken as 1/10; the adjacent span is limited to a range between $\ngtr 0.833\ l'$ to $\nless 1.20\ l'$. With varying span lengths, different amounts of steel will be bent up. This computation is based upon the assumption that the adjacent span = 1.00 l' and the trussed bar in it is also 1-#11. Then:—

$$-M = -wl'^2/10$$

Since the added top bar increases A_s over that used in "positive flexure" above, the beam is over-reinforced and compressive steel will be needed, obtained by lapping the bottom bars 20 diameters past each other, thus affording 2-#9 for compression. As shown below, compute x by taking moments about the bottom of the beam and doubling A'_s as per ACI 706b:—(See figure on page 221.)

$$x = \frac{\frac{12x^2}{2} + 38.00 \times 2.54 + 41.2 \times 21.82}{12x + 38.00 + 41.2}$$

$$6x^2 + 79.20x = 96.52 + 898.98 = 995.50$$

$$x^2 + 13.20 + (6.60)^2 = 165.92 + 43.56 = 209.48$$

$$x = 6.60 \pm 14.47 = 7.87 \text{ in.}$$

$$C_c = 12 \times 7.87 \times \frac{1350}{2} = 63{,}750 \text{ lb}$$

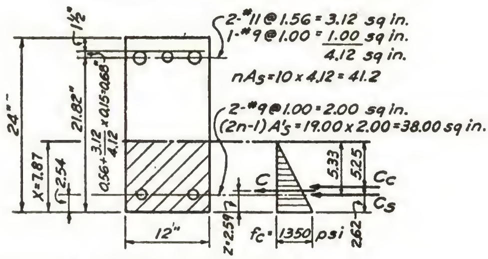
$$C_s = 38 \times 1350 \times \frac{5.33}{7.87} = 34{,}740 \text{ lb}$$

$$C = C_c + C_s \qquad = 98{,}490 \text{ lb}$$

$$z = 2.54 + \frac{63.75}{98.49} \times 0.08 = 2.59 \text{ in.}; \qquad jd = 21.82 - 2.59 = 19.23 \text{ in.}$$

$$M_c = (C_c + C_s)jd = 98{,}490 \times 19.23 = 1{,}894{,}000 \text{ lb-in.}$$

REINFORCED CONCRETE BEAMS
CONTINUOUS END SPAN

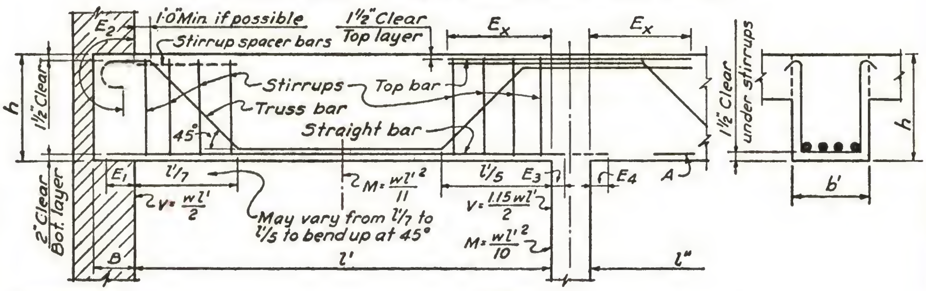


$M_t = 4.12 \times 20{,}000 \times 19.23 = 1{,}585{,}000$ lb-in.

$$w = \frac{-M}{0.1\,wl'^2} = \frac{1{,}585{,}000}{0.1 \times 20 \times 20 \times 12} = 3300 \text{ plf} > 3020 \text{ plf from positive flexure.}$$

These computations indicate the great range of choices open to the designer, the impracticability of anything approaching a complete set of beam tables, and the fact that continuity not only affects the moments and shears in this span but the amount of steel brought through from the adjacent span. Hence these tables can only serve for making preliminary estimates of sizes, and any major structure should be designed by computation.

FOR THE TABLE ON PAGES 222 AND 223:—



STRESSES:—

f_s = 20,000 psi
f'_c = 3000 psi
f_c = 1350 psi
v_c = 90 psi
v = 360 psi
u = 300 psi for bottom bars
= 210 psi for top bars with over 12″ of concrete under them

CODES:—

"Building Code Requirements for Reinforced Concrete (ACI 318-56)," also "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

E_1 = 6 in. minimum for bottom bars.

E_2 = 17 bar diameters, usually requiring a semicircular hook. (24 bar diameters for depths over 12 in.)

E_3 = bottom bar to extend 6 in. into the support except when values in the load tables are printed in boldface type.

E_4 = When the values in the load tables are printed in boldface type, bottom bars must lap bars of adjoining span 20 diameters so that they may serve for compressive reinforcement, as shown in example in the text.

$E_x = l'/4$ or $l''/4$ or 17 bar dias. (24 dias., $d > 12$ in.) past bend-down point, whichever is greatest

ACI 902a requires the extending of top bars past innermost position of point of inflection $l/16$, d, or half-bond length; if point of inflection is at $l/5$ of center-to-center span, if beam depth is $l/12$, and if column face is $l/15$, the given ratios, which are easily applied, work out fairly well, but must be checked for actual use.

B = Ordinarily 8 in. minimum and sufficient in any case to keep the bearing pressure on the wall within the allowable for the material of which the wall is made. (See page 99.)

A = Bars in adjoining span, not shown.

For stirrup combinations, see page 230.

CONTINUOUS REINFORCED CONCRETE BEAMS—END SPAN

For limitations and explanation of use of this table, see page 221.

	Stem		Bar Combinations							TOTAL SAFE CARRYING CAPACITY * (plf)— UNIFORMLY DISTRIBUTED; Span l' in Feet							
			Straight		Trussed			Top		10		12		14		16	
	h (in.)	b′ (in.)	No.	Size	No.	Size	No. Layers	No.	Size	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load	Stir-rup Mark	Safe Load
1	12	6	2	#4	1	#5	2	1	#4	28a	1028	25a	714	—	524	—	401
2	12	8	1	#6	1	#7	1	—	—	24a	1159	24a	964	24a	795	—	609
3	12	10	2	#5	2	#5	1	1	#4	28a	1931	25a	1341	23a	986	—	755
4	14	8	1	#7	1	#8	1	1	#4	26b	1659	27a	**1380**	27a	**1183**	26b	**910**
5	14	10	1 1	#6 #5	1	#8	1	1	#3	28b	2614	28b	1980	27a	1459	26b	1115
6	14	12	2	#6	2	#7	1	1	#4	28b	2853	28b	2375	27a	1810	26b	1385
7	16	8	2	#5	1	#8	2	1	#4	37b	2620	28d	1890	27b	1460	26c	1120
8	16	10	2	#6	1	#9	1	1	#4	37b	3254	38d	2660	28d	1952	26c	1497
9	16	12	1 1	#7 #6	2	#7	1	1	#6	37b	3629	38d	3020	210c	2480	28d	**1900**
10	18	8	1 1	#6 #5	2	#6	2	1	#5	38e	3328	38e	2659	29d	1950	28e	1492
11	18	10	2	#6	2	#7	2	1	#4	37d	3625	38e	3020	38e	2435	29d	1861
12	18	12	2	#7	2	#8	1	1	#4	37d	4468	38e	3720	38e	3190	38e	**2476**
13	20	8	2	#6	1	#9	2	1	#4	39f	4118	39f	3345	38f	2460	29f	1880
14	20	10	1 1	#7 #6	2	#7	2	1	#6	37e	4577	39g	3820	39g	3119	38f	2400
15	20	12	2	#7	2	#8	1	1	#6	36f	5042	38e	4200	38f	3600	38f	3040
16	22	8	2	#6	2	#7	2	1	#4	37g	4589	39g	3825	39i	3120	39i	2385
17	22	10	2 1	#6 #5	3	#6	2	1	#8	310g	6527	311f	5225	39i	3840	38h	2940
18	22	12	3	#6	3	#7	2	1	#6	39h	6884	311f	5725	311f	4675	39i	3580
19	24	10	3	#6	2	#8	2	1	#6	311f	7635	313f	6350	312j	4792	310l	3665
20	24	12	2	#9	1	#11	1	1	#9	39h	7928	311f	6600	312j	5660	312j	4650
21	24	14	3	#7	3	#8	2	1	#5	310j	8891	312i	7400	313f	6350	312j	5080
22	26	10	2 2	#6 #5	2	#8	2	1	#8	314a	10253	314b	7775	313i	5710	311m	4375
23	26	12	3	#7	2	#9	2	1	#7	312i	9748	314b	8110	315a	6850	313i	5240
24	26	14	3	#8	2	#10	2	1	#7	312m	11067	314b	9200	316a	7900	315a	6045
25	28	12	4	#6	3	#8	2	1	#6			316b	10040	316b	8070	315a	6180
26	28	14	3	#8	2	#10	2	1	#8			314e	10000	316b	8600	316c	7145
27	28	16	4	#7	4	#8	2	1	#6			411o	11720	412n	10050	412p	8199
28	30	12	5	#6	4	#7	2	1	#9					319a	9530	317e	7300
29	30	14	5	#6	4	#8	2	1	#6					414f	11100	413o	8500
30	30	16	5	#7	4	#8	2	1	#9					414j	12510	414k	9600

* Deduct dead load from these values.
Where dash occurs in stirrup column, no stirrups are required.
Extend bottom bars to E_4 (page 221) for values in boldface type.

CONTINUOUS REINFORCED CONCRETE BEAMS—END SPAN

TOTAL SAFE CARRYING CAPACITY* (plf)—
UNIFORMLY DISTRIBUTED

Span l' in Feet

18		20		22		24		26		28		30		
Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	
—	317	—	257	—	212	—	178							1
—	481	—	389	—	322	—	270							2
—	596	—	483	—	399	—	336							3
23b	720	—	582	—	481	—	405	—	345	—	297			4
23b	882	—	714	—	590	—	496	—	422	—	364			5
23b	1093	—	885	—	732	—	615	—	524	—	452			6
25b	884	23c	715	—	591	—	497	—	423	—	365	—	318	7
25b	1181	23c	956	—	790	—	664	—	565	—	488	—	425	8
26c	1500	24b	1215	—	1005	—	844	—	719	—	619	—	540	9
26e	1180	25c	954	24c	790	—	664	—	565	—	488	—	424	10
27d	1470	25c	1191	24c	985	—	827	—	705	—	608	—	530	11
28e	1955	26e	1585	25c	1310	23d	1100	—	937	—	808	—	704	12
28f	1487	27f	1205	26e	995	24d	837	23d	712	—	615	—	535	13
37f	1900	27e	1535	26e	1270	24d	1068	23d	908	—	783	—	682	14
37f	2404	28e	1950	26f	1610	24d	1352	23d	1152	—	995	—	865	15
39i	1888	29g	1530	27h	1263	26g	1060	25e	904	24d	780	23d	679	16
38h	2320	28h	1880	27g	1555	26g	1306	24e	1112	23e	960	—	835	17
38h	2830	210g	2290	28h	1895	26g	1590	25e	1355	24d	1170	23e	1008	18
39l	2900	212j	2345	210l	1940	28j	1630	27h	1388	26g	1198	24e	1030	19
310l	3680	39l	3020	212j	2462	29i	2070	27j	1762	26g	1520	25e	1325	20
310l	4010	39l	3255	212j	2690	29i	2260	27j	1925	26g	1660	24e	1448	21
310p	3460	39n	2800	212n	2318	210l	1947	28l	1657	27h	1430	27h	1245	22
311m	4140	310p	3354	39n	2770	212n	2330	29m	1984	27k	1711	27h	1490	23
313i	4780	311m	3865	39n	3200	39n	2682	210l	2285	28j	1975	26h	1700	24
313i	4880	312p	3960	310s	3270	39p	2750	212n	2340	211o	2020	28l	1760	25
314f	5645	313i	4560	311p	3780	39p	3165	213i	2703	211o	2330	29m	2030	26
315a	6455	314f	5245	312p	4330	310s	3640	39n	3100	212n	2678	29m	2330	27
315d	5760	314k	4660	312r	3860	311r	3245	39q	2762	213o	2382	211o	2078	28
317e	6700	315d	5430	314k	4482	312r	3770	310s	3220	215d	2775	212p	2417	29
318a	7580	316g	6140	314k	5070	313o	4260	311r	3630	39p	3119	213i	2730	30

$d = h - (2'' +$ distance from bottom of bars to their centroid).

REINFORCED CONCRETE BEAMS

(WORKING LOAD METHOD)

CONTINUOUS INTERIOR SPANS

The table on page 228 gives the total safe uniform load per lineal foot (live and dead) * on reinforced concrete beams for the interior spans only of continuous runs of beams, computed in conformity with the American Concrete Institute's "Building Code Requirements for Reinforced Concrete (ACI 318-56)"; for span lengths varying from 10 to 30 feet in two-foot multiples; for one set of stresses, viz. $f_s = 20{,}000$ psi and $f_c = 1350$ psi; for depths from 12 to 30 inches in two-inch multiples; for three widths (b', page 227) in each depth; with bar combinations to produce balanced reinforcement ($p = 0.0136$) for a rectangular beam of the size given.

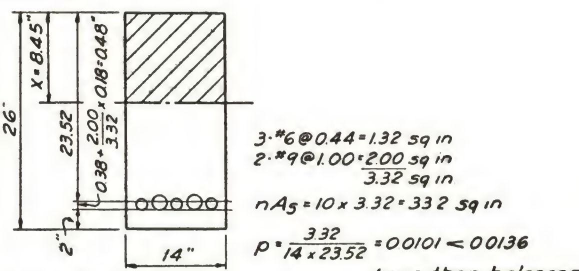
There is no great advantage in listing much more heavily reinforced continuous beams, because negative bending quite sharply limits their maximum capacity. Following the argument developed for this same situation for the continuous end of an end span, it can be shown that the limit is nearer $\left(\dfrac{11 \times 450}{16 \times 235 \times (1.1)^2}\right) = 1.09$ than 1.43 (see page 218).

The limitations and arrangement of the following table parallel so closely those of the single span that pages 210 to 213 should be studied carefully before reading further. This applies especially to stirrup arrangements and bond, and to the fact that bars must meet ASTM A305, and that carrying capacity is taken as the least of the limits set by shear, bond, positive or negative flexure.

The following example may prove useful for those who wish to design beyond the scope of the tables or to see how they were prepared:—

Example—For the table on page 228-229, determine the safe carrying capacity on an interior span (of a continuous run) of 22 feet of a 14 × 26 in. beam stem reinforced with 3-#6 bottom bars, 2-#9 truss bars, all in one layer, and 1-#8 top bar over each support, assuming 2-#9 truss bars bent up and extended through from adjacent span. Check the stirrup combination 38l.

Solution—From the figure determine first the distance "x" down to the neutral axis:—



$$\frac{14x^2}{2} = 33.2\,(23.52 - x)$$

$$7x^2 + 33.2x = 780.86$$

$$x^2 + 4.74x + (2.37)^2 = 111.55 + 5.62 = 117.17$$

$$x = -2.37 \pm 10.82 = 8.45 \text{ in. } (0.36d)$$

* In the various slab tables throughout this book, the weight of the slab has been deducted so that the values given in the tables are the safe superimposed loads. In the case of beams, there is no advantage in deducting the minor weight of the beam stem, as it is the weight of the tributary slab that is the main element of dead load. So in these tables the capacity given is the total safe load, dead plus live.

REINFORCED CONCRETE BEAMS
CONTINUOUS INTERIOR SPANS

Positive Flexure—The arm of the internal couple, $jd = 23.52 - \frac{8.45}{3} = 20.7$ in.

$$\left.\begin{aligned} M_s &= A_s f_s jd = 3.32 \times 20{,}000 \times 20.7 = 1{,}375{,}000 \text{ lb-in.} \\ M_c &= bkd\frac{f_c}{2}jd = 14 \times 8.45 \times \frac{1350}{2} \times 20.7 = 1{,}655{,}000 \text{ lb-in.} \end{aligned}\right\} \text{a somewhat under-reinforced beam.}$$

$$w = \frac{M}{l'^2\frac{12}{16}} = \frac{1{,}375{,}000}{22 \times 22 \times \frac{12}{16}} = 3790 \text{ plf (3720 in table)}$$

Bond on Bottom Bars—The bond on the bottom bars is to be computed in that part of the positive bending zone which has the highest external shear, i.e., the point of inflection. Since the maximum positive moment is $wl'^2/16$, the point of inflection is 0.147 l' from the support (chart, page 84), at which point the shear is approximately 70 per cent of $wl'/2$.

$V = \Sigma ojdu = 3 \times 2.356 \times 20.7 \times 300 = 43{,}900$ lb

$$w = \frac{43{,}900}{0.70 \times 11} = 5700 \text{ plf} > 3790 \text{ plf, or more than allowed by flexure.}$$

Shear—For spacing stirrups, it is not practicable to work backwards, so take the capacity of the beam at 3790 plf as computed for positive bending and design the web reinforcement to take care of this: *—

$$V = \frac{wl'}{2} = 3790 \times 11 = 41{,}690 \text{ lb}$$

$$v = \frac{V}{bjd} = \frac{41{,}690}{14 \times 20.7} = 144 > 90 < 360 \text{ psi}$$

Deducting $v_c = 90$ psi leaves 54 psi to be carried by stirrups.
Compute the distance to be covered by stirrups:—

$$a = \frac{54}{144} \times 11 \times 12 = 49\tfrac{1}{2} \text{ in.}$$

The total area of stirrups:—

$$A_v = \frac{bav_s}{2f_v} = \frac{14 \times 49\frac{1}{2} \times 54}{2 \times 20{,}000} = 0.935 \text{ sq in.}$$

Minimum requirement is 5-#3 U-stirrups = 1.10 sq in.
The spacing is figured by the method explained on pages 86-91 as 2½, 5½, 6½, 8, 11½, without exceeding $d/2$ (ACI 806a). ACI 801d requires that stirrups cover a distance $(a + d)$, spaced at not over $d/2$, so use 7-#3 U-stirrups, spaced 3, 5, 6, 8, 11, 12, 12. The combination furnished in the table is 38l, or #3 stirrups, 8 at each end of the span, spaced 2, 6, 6, 7, 11, 11, 11, 11, which is about as close as can be expected with a standardized group.
Had this been a continuous or restrained beam or frame without a monolithic slab to provide T-beam action, additional web reinforcement would have had to be provided as per ACI 801e.

Negative Flexure—By Code, the negative moment is to be taken as $wL^2_{av}/11$, and the adjacent span is limited to a range between $\gtrless$ 0.833 l' and $\lessgtr$ 1.20 l'. With varying span lengths, different amounts of steel will be bent up. This computation is based upon the assumption that the adjacent span = 1.00 l' and the truss bars in it are also 2-#9. Then:—

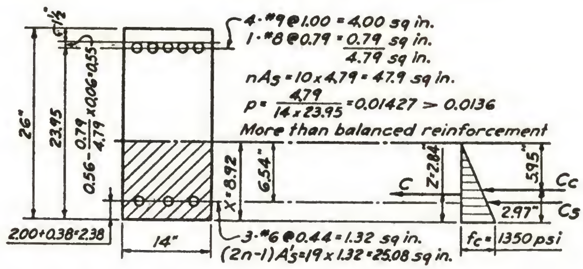
$$-M = -wl'^2/11$$

---

* Assuming live load over one-half the span, the maximum live shear should not be taken as zero at midspan but as $wl'/8$ (one-quarter of the end shear); against this the bent-up bars provide some web reinforcement.

REINFORCED CONCRETE BEAMS
CONTINUOUS INTERIOR SPANS

Since the added top bar increases A_s over the area used in "positive flexure," and since p is quite a bit over 0.0136, the beam is overreinforced and compressive steel is needed, obtained by lapping the bottom bars 20 diameters past each other, thus affording 3-#6 for compression. From the figure below, compute x by taking moments about the bottom of the beam and doubling A'_s as per ACI 706b:—



$$x = \frac{\frac{14x^2}{2} + (25.08 \times 2.38) + (47.9 \times 23.95)}{14x + 23.76 + 47.9}$$

$$7x^2 + 72.98x = 59.69 + 1147.21 = 1206.9$$
$$x^2 + 10.43x + (5.22)^2 = 172.41 + 27.25 = 199.66$$
$$x = -5.22 \pm 14.14 = 8.92 \text{ in.}$$
$$C_c = \frac{8.92 \times 1350 \times 14}{2} = \quad 84{,}300 \text{ lb}$$
$$C_s = 25.08 \times \frac{6.54}{8.92} \times 1350 = \underline{24{,}800 \text{ lb}}$$
$$C = C_c + C_s \qquad = 109{,}100 \text{ lb}$$
$$z = 2.38 + \frac{84.3}{109.1} \times \left(\frac{8.92}{3} - 2.38\right) = 2.84 \text{ in.}; \qquad jd = 23.95 - 2.84 = 21.1 \text{ in.}$$
$$M_c = 108{,}150 \times 21.1 \qquad = 2{,}283{,}000 \text{ lb-in.}$$
$$M_s = 4.79 \times 20{,}000 \times 21.1 = 2{,}022{,}000 \text{ lb-in.}$$
$$w = \frac{2{,}022{,}000 \times 11}{22 \times 22 \times 12} = 3830 \text{ plf} > 3790 \text{ plf, so positive moment}$$

governs (when $l'' \lessgtr l'$).

Bond on Negative Bars—The maximum end shear:—

$$V = 3790 \times 11 = 41{,}690 \text{ lb}$$

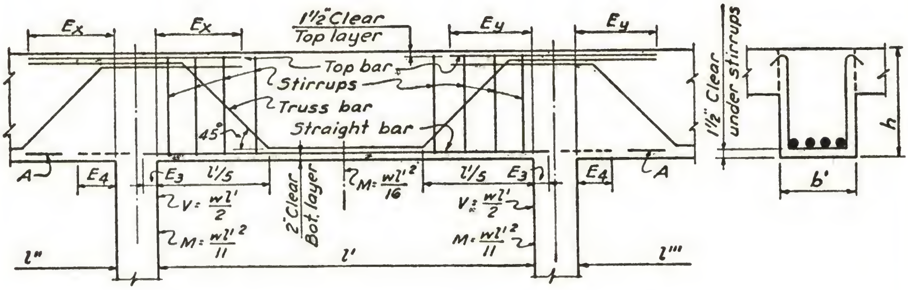
This is taken by 4-#9 + 1-#8 bars. $\frac{4.00}{4.79}$ or 34,800 lb is taken by the #9 bars and $\frac{0.79}{4.79}$ or 6890 lb is taken by the #8 bar. Then $u = \frac{V}{\Sigma ojd} = \frac{34{,}800}{(4 \times 3.544)21.1} = 117 \text{ psi} < 210 \text{ psi}$ on the #9 bars and $\frac{6890}{3.142 \times 21.1} = 104$ psi on the #8 bar, so the computed shear is well within the allowable for top bars.

Comment—The bending and extension of truss bars, possible staggering of bends, length of top bar, extension or lap of bottom bars, and so on can be computed, but the foregoing is sufficient to illustrate the make-up of the tables.

Under the extreme case of $l'' = 1.20\, l'$, it would be necessary to check negative flexure to make sure both that the extra top bar combined with the truss bars from the two spans provides sufficient tension steel (adding extra top steel if needed) and that the bottom bars provide sufficient compression steel (extending the bars through the support if necessary to increase the area available and carrying them out to a point where the concrete alone provides adequate compression resistance).

CONTINUOUS REINFORCED CONCRETE BEAMS—INTERIOR SPANS

Applies to the tables on pages 228 and 229.



STRESSES:—

f_s = 20,000 psi
f'_c = 3000 psi
f_c = 1350 psi
v_c = 90 psi

v = 360 psi
u = 300 psi for bottom bars
= 210 psi for top bars with over 12″ of concrete under them

CODES:—

"Building Code Requirements for Reinforced Concrete (ACI 318-56)," also "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)."

E_1 = 6 in. minimum for bottom bars.

E_2 = 17 bar diameters (24 diameters when $d > 12$ in.), usually requiring a semicircular hook.

E_3 = bottom bar to extend 6 in. into the support except when values in the load tables are printed in boldface type.

E_4 = When the values in the load tables are printed in boldface type, bottom bars must lap bars of adjoining span 20 diameters so that they may serve for compressive reinforcement, as shown in the example in the text.

$$E_x = \left\{\begin{array}{l} \dfrac{l'}{4} \\ \dfrac{l''}{4} \\ \text{24 bar diameters} \end{array}\right\} \text{whichever is greatest}$$

ACI 902a requires the extending of top bars past the innermost position of the point of inflection $l/16$, d, or half-bond length; if point of inflection is at $l/5$ of center-to-center span, if beam depth is $l/12$, and if column face is $l/15$, the given ratios, which are easily applied, work out fairly well, but must be checked for actual use.

$$E_y = \left\{\begin{array}{l} \dfrac{l'}{4} \\ \dfrac{l'''}{4} \\ \text{17 bar dias. (24} \\ \text{dias., } d > 12 \text{ in.)} \\ \text{past bend-down} \\ \text{point} \end{array}\right\} \text{whichever is greatest}$$

A = Bars in adjoining span, not shown.

For stirrup combinations, see page 230.

CONTINUOUS REINFORCED CONCRETE BEAMS—INTERIOR SPANS

For limitations and use of this table, see page 227.

	Stem		Bar Combinations							TOTAL SAFE CARRYING CAPACITY* (plf)—UNIFORMLY DISTRIBUTED							
										Span l' in Feet							
			Straight		Trussed			Top		10		12		14		16	
	h (in.)	b′ (in.)	No.	Size	No.	Size	No. Layers	No.	Size	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load
1	12	6	2	#3	1	#5	2	1	#3	26*a*	1128	23*a*	734	—	539	—	413
2		8	2	#3	1	#6	1	1	#3	25*a*	1481	23*a*	1030	—	756	—	579
3		10	2	#4	2	#5	1	1	#3	26*a*	2115	25*a*	1470	—	1080	—	826
4	14	8	1	#6	1	#7	1	1	#5	26b	2041	27*a*	1700	27*a*	**1430**	26b	**1096**
5		10	2	#4	1	#8	1	1	#5	27*a*	2726	27*a*	2225	26b	**1638**	25*a*	**1253**
6		12	2	#5	2	#6	1	1	#6	28c	3417	28c	2820	28c	2070	26*a*	1588
7	16	8	2	#4	1	#7	1	1	#5	28c	3154	26*d*	2190	25c	1610	24c	1231
8		10	1 1	#5 #4	1	#8	1	1	#5	28c	3588	28c	2790	25c	2050	24c	1570
9		12	2	#5	2	#6	1	1	#6	28c	4005	28c	3305	25c	2430	24c	1860
10	18	8	1 1	#5 #4	2	#5	2	1	#6	37c	3903	37c	2715	27c	1990	26*d*	1526
11		10	2	#5	2	#6	1	1	#6	37c	4587	37c	3685	27c	2710	26*d*	2075
12		12	1 1	#6 #5	2	#7	1	1	#4	36*d*	5027	37c	4190	37c	**3310**	26*d*	**2530**
13	20	8	2	#4	2	#6	2	1	#4	37c	3930	37e	3280	37e	2539	27e	1940
14		10	2	#5	2	#6	1	1	#6	37c	5171	37e	4260	36*f*	3130	27c	2400
15		12	2	#6	2	#7	1	1	#7	37c	6172	37e	5140	38*f*	**4300**	37e	**3290**
16	22	8	2	#5	2	#6	2	1	#5	37e	5503	38*f*	4320	37*h*	3180	37*h*	2432
17		10	4	#4	3	#5	2	1	#8	39*h*	7616	38*g*	5295	38*f*	3890	28*g*	2980
18		12	2	#6	2	#7	1	1	#8	37c	6870	37e	5720	38*f*	4800	37*h*	3672
19	24	10	3	#5	2	#7	2	1	#6	310*j*	9162	310*k*	6575	38*j*	4830	37*j*	3700
20		12	3	#6	1	#10	1	1	#9	312*i*	11325	312*i*	8450	310*k*	6210	39*k*	4750
21		14	3	#6	3	#7	1	1	#7	310*j*	11346	312*i*	9440	311*h*	7375	39*k*	5650
22	26	10	3	#5	2	#7	2	1	#8	311*h*	10035	311*h*	7820	310*k*	5750	38*j*	4400
23		12	3	#5	2	#8	2	1	#7	39*h*	10035	311*h*	8370	311*h*	6780	39*k*	5199
24		14	3	#6	2	#9	1	1	#8	310*n*	12382	312*i*	10300	313*h*	8840	312*n*	**7030**
25	28	12	4	#5	3	#7	2	1	#7			314e	11540	313*l*	8495	312*n*	6500
26		14	3	#6	2	#9	1	1	#8			311*h*	10710	311*q*	9200	311*l*	7325
27		16	4	#5	4	#7	2	1	#7			312*i*	12100	314e	10400	313*h*	8175
28	30	12	5	#5	4	#6	2	1	#9					314*i*	9900	313*n*	7570
29		14	5	#5	4	#7	2	1	#8					318*d*	12200	316*f*	9330
30		16	5	#6	4	#7	2	1	#9					317*d*	12610	315c	9660

* Deduct dead load from these values.
Where dash occurs in stirrup column, no stirrups are required.
Extend bottom bars to E_4 (page 227) for values in boldface type.

CONTINUOUS REINFORCED CONCRETE BEAMS—INTERIOR SPANS

TOTAL SAFE CARRYING CAPACITY * (plf)—
UNIFORMLY DISTRIBUTED

Span l' in Feet

18		20		22		24		26		28		30		
Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	Stirrup Mark	Safe Load	
—	326	—	264	—	218	—	183							1
—	457	—	370	—	306	—	257							2
—	652	—	528	—	436	—	367							3
24a	867	—	701	—	580	—	487	—	415	—	358			4
—	990	—	802	—	662	—	556	—	474	—	408			5
23c	1252	—	1015	—	839	—	705	—	600	—	517			6
23c	973	—	787	—	651	—	546	—	466	—	402	—	350	7
23c	1240	—	1000	—	830	—	697	—	543	—	512	—	446	8
23c	1470	—	1190	—	984	—	825	—	704	—	606	—	529	9
24c	1206	23c	976	—	807	—	678	—	577	—	498	—	434	10
25c	1640	24c	1328	—	1100	—	922	—	786	—	677	—	590	11
26d	2000	24c	1625	23c	1341	—	1128	—	960	—	828	—	721	12
26f	1534	25c	1240	24c	1028	—	864	—	735	—	635	—	552	13
26d	1892	24d	1534	23c	1268	—	1065	—	907	—	783	—	682	14
27e	2600	26f	2105	25c	1740	24d	1462	—	1245	—	1075	—	935	15
27e	1920	26f	1558	25e	1288	24d	1080	23d	920	—	793	—	690	16
27e	2352	26f	1905	25e	1575	23d	1324	—	1128	—	972	—	845	17
27e	2900	27c	2350	24d	1942	23d	1630	—	1390	—	1200	—	1045	18
37h	2900	28f	2365	26h	1955	26f	1645	24f	1400	23d	1208	—	1050	19
37j	3760	37j	3040	27j	2520	26h	2115	25f	1800	24f	1550	23d	1350	20
38j	4460	210k	3615	28g	2990	26h	2510	25f	2140	24f	1845	23d	1608	21
37k	3480	29k	2820	28j	2330	26h	1958	25f	1668	24e	1440	23e	1251	22
38j	4100	37k	3320	28k	2742	26h	2310	25f	1965	24e	1696	23e	1479	23
311l	5550	39m	4500	38l	3720	37k	3130	28k	2660	26h	2300	25f	2000	24
310r	5140	39m	4160	38l	3440	210k	2890	28k	2460	27k	2121	25g	1850	25
310r	5790	38l	4680	37k	3875	29k	3260	27l	2775	25g	2390	24e	2083	26
311l	6450	39m	5230	38l	4325	210k	3635	28k	3100	25g	2672	24e	2325	27
312q	5985	310t	4848	39r	4000	38m	3365	210r	2870	28l	2475	27k	2155	28
314j	7380	313n	5975	310t	4940	310t	4150	38m	3540	37n	3050	37n	2660	29
313n	7630	311l	6180	310r	5110	38m	4300	37n	3660	27m	3160	27k	2750	30

$d = h - (2'' +$ distance from bottom of bars to their centroid).

STIRRUP COMBINATIONS

The method of computing stirrups is illustrated in the examples on pages 212, 220 and 225 and explained in detail on pages 86 ff.

For the beam tables in this section, certain combinations of stirrups were standardized. With uniform loads, the spacing will theoretically change with every variation in length of span, but these combinations were selected as providing sufficient web reinforcement without being too wasteful of material.

Each combination is identified both in the table of carrying capacity and in the table of stirrup combinations by a numerical designation or "mark." The first digit in the mark in the table of carrying capacity gives the numerical bar size of which the stirrup is made and does not appear in the table of stirrup combinations below. The second (and possible third) digit gives the number of U-shaped stirrups in each end of the beam. The final letter indicates the stirrup spacing as given in the following table.

Spacings are given from face of support towards center of span.

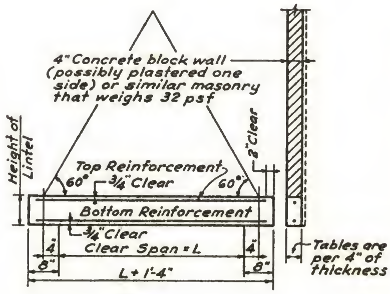
Mark	Spacing on each end	Mark	Spacing on each end	Mark	Spacing on each end
3a	2 4 4	9h	1 3 4 4 5 7 9 9 9	12m	1 2 2 3 3 3 3 4 5 7 11 11
3b	3 5 5	9i	2 4 5 6 7 9 9 9 9	12n	1 3 3 4 4 5 5 7 9 11 11 11
3c	2 6 6	9j	2 6 7 9 9 9 9 9 9	12o	2 5 6 6 6 9 11 11 11 11 11 11
3d	4 8 8	9k	2 4 4 4 6 9 10 10 10	12p	2 4 4 4 5 6 6 8 11 12 12 12
3e	6 9 9	9l	2 5 6 7 9 10 10 10 10	12q	1 4 4 4 4 5 5 7 10 13 13 13
4a	2 5 5 5	9m	2 4 5 6 7 11 11 11 11	12r	2 4 5 5 5 6 8 8 13 13 13 13
4b	2 6 6 6	9n	2 6 6 8 9 11 11 11 11	12s	2 6 6 6 6 8 9 11 13 13 13 13
4c	2 7 7 7	9o	2 4 4 5 7 9 12 12 12	13a	1 2 3 3 3 4 4 5 6 6 6 6 6
4d	4 8 8 8	9p	2 6 7 8 10 12 12 12 12	13b	1 2 3 3 4 4 4 6 7 7 7 7 7
4e	6 9 9 9	9q	2 6 7 7 8 13 13 13 13	13c	1 2 3 3 4 4 4 5 7 8 8 8 8
4f	3 8 10 10	10a	1 4 4 4 4 4 4 4 4 4	13d	1 3 4 4 5 5 5 7 8 8 8 8 8
5a	3 4 4 4 4	10b	1 3 4 4 5 5 5 5 5 5	13e	1 4 4 5 5 6 6 8 9 9 9 9 9
5b	2 6 6 6 6	10c	1 4 4 5 6 6 6 6 6 6	13f	1 3 3 3 3 4 4 5 6 8 10 10 10
5c	2 7 7 7 7	10d	2 5 6 6 6 6 6 6 6 6	13g	1 4 4 4 4 5 5 7 9 10 10 10 10
5d	2 8 8 8 8	10e	2 6 7 7 7 7 7 7 7 7	13h	1 2 3 3 3 4 4 4 6 8 11 11 11
5e	6 9 9 9 9	10f	2 5 7 8 8 8 8 8 8 8	13i	1 3 3 4 4 4 5 5 7 10 11 11 11
5f	3 8 10 10 10	10g	1 3 5 5 6 6 8 9 9 9	13j	1 4 4 5 5 6 6 6 9 11 11 11 11
5g	3 7 10 12 12	10h	2 4 5 6 6 9 9 9 9 9	13k	2 5 5 5 6 6 8 8 11 11 11 11 11
6a	2 4 4 4 4 4	10i	2 5 7 7 9 9 9 9 9 9	13l	1 2 3 3 3 3 3 4 5 5 8 12 12
6b	2 5 5 5 5 5	10j	1 2 3 3 4 4 5 7 10 10	13m	1 5 5 5 5 5 7 7 10 12 12 12 12
6c	2 6 6 6 6 6	10k	1 3 4 4 5 6 8 10 10 10	13n	1 3 3 3 4 4 4 5 5 9 13 13 13
6d	2 5 7 7 7 7	10l	2 4 5 5 6 7 10 10 10 10	13o	1 4 4 4 4 5 5 6 8 11 13 13 13
6e	4 7 7 7 7 7	10m	3 7 7 9 10 10 10 10 10 10	13p	2 5 5 5 6 6 7 8 10 13 13 13 13
6f	3 6 8 8 8 8	10n	1 2 2 3 3 4 5 6 11 11	14a	1 2 2 2 3 3 3 3 3 4 5 8 11 11
6g	4 7 9 9 9 9	10o	1 3 4 4 4 5 8 11 11 11	14b	1 2 3 3 3 3 3 3 4 5 6 8 11 11
6h	2 6 10 10 10 10	10p	2 4 5 6 8 8 11 11 11 11	14c	1 3 4 4 4 4 5 5 6 8 11 11 11 11
7a	2 4 5 5 5 5 5	10q	3 7 8 9 11 11 11 11 11 11	14d	1 4 4 4 5 5 5 6 6 9 11 11 11 11
7b	2 6 6 6 6 6 6	10r	1 4 4 5 5 6 10 12 12 12	14e	1 2 2 2 2 3 3 3 4 5 5 7 12 12
7c	2 4 6 7 7 7 7	10s	2 5 5 7 7 9 12 12 12 12	14f	1 3 3 3 4 4 4 4 5 6 7 10 12 12
7d	3 6 7 7 7 7 7	10t	1 4 5 5 5 7 9 13 13 13	14g	1 4 4 4 5 5 5 6 7 9 12 12 12 12
7e	2 5 6 8 8 8 8	11a	1 3 3 4 4 5 5 5 5 5 5	14h	2 4 4 5 5 6 6 7 8 10 12 12 12 12
7f	3 7 8 8 8 8 8	11b	2 5 5 5 5 5 5 5 5 5 5	14i	1 2 2 3 3 3 3 3 4 5 5 8 13 13
7g	2 6 7 9 9 9 9	11c	2 4 4 5 6 7 7 7 7 7 7	14j	1 2 3 3 3 3 4 4 4 6 6 10 13 13
7h	3 8 9 9 9 9 9	11d	1 3 4 5 5 6 6 8 8 8 8	14k	1 3 3 4 4 4 4 5 5 7 11 13 13 13
7i	2 5 7 9 10 10 10	11e	2 6 6 7 8 8 8 8 8 8 8	15a	1 2 3 3 3 3 4 4 4 5 6 9 11 11 11
7j	2 6 8 10 10 10 10	11f	1 3 3 4 4 6 6 8 9 9 9	15b	1 3 3 3 4 4 4 5 5 5 7 11 11 11 11
7k	2 6 8 11 11 11 11	11g	2 5 5 7 7 9 9 9 9 9 9	15c	1 2 2 2 3 3 3 3 3 4 4 6 8 13 13
7l	2 5 7 9 12 12 12	11h	1 2 3 3 4 4 5 7 10 10 10	15d	1 3 3 3 3 4 4 4 5 5 6 7 10 13 13
7m	2 4 6 7 10 13 13	11i	1 3 4 4 5 5 6 10 10 10 10	15e	1 4 4 4 5 5 6 6 6 8 9 13 13 13 13
7n	3 6 8 11 13 13 13	11j	2 4 5 6 6 6 9 10 10 10 10	16a	1 2 2 2 3 3 3 3 3 4 4 4 6 8 11 11
8a	2 4 4 4 4 4 4 4	11k	2 6 6 6 8 10 10 10 10 10 10	16b	1 2 2 2 3 3 3 3 3 4 4 5 6 8 12 12
8b	1 4 5 5 5 5 5 5	11l	1 3 4 4 5 5 7 9 11 11 11	16c	1 2 2 3 3 3 3 3 4 4 5 5 6 9 12 12
8c	1 3 4 5 5 6 6 6	11m	2 4 4 5 6 8 8 11 11 11 11	16d	1 3 3 3 4 4 4 4 5 5 6 8 11 12 12 12
8d	2 5 6 6 6 6 6 6	11n	2 6 6 8 8 11 11 11 11 11 11	16e	1 3 4 4 4 4 5 5 5 7 7 9 12 12 12 12
8e	2 5 7 7 7 7 7 7	11o	1 4 4 4 4 6 7 10 12 12 12	16f	1 2 2 2 2 3 3 3 3 4 4 4 6 8 13 13
8f	2 6 7 8 8 8 8 8	11p	2 4 5 5 6 7 9 12 12 12 12	16g	1 2 3 3 3 3 4 4 4 4 5 6 7 10 13 13
8g	2 4 5 5 8 9 9 9	11q	1 3 3 3 4 4 4 6 8 13 13	17a	1 2 2 3 3 3 3 3 4 4 4 5 6 8 11 11 11
8h	2 5 7 7 9 9 9 9	11r	2 5 5 5 7 7 9 13 13 13 13	17b	1 2 3 3 3 3 4 4 4 4 5 5 7 10 12 12 12
8i	2 4 4 6 9 10 10 10	12a	1 2 3 3 4 4 4 4 4 4 4 4	17c	1 3 4 4 4 4 5 5 5 5 7 7 9 12 12 12 12
8j	2 5 6 7 10 10 10 10	12b	1 2 3 3 3 4 5 5 5 5 5 5	17d	1 1 2 2 2 2 2 3 3 3 3 3 4 5 7 13 13
8k	2 4 5 6 9 11 11 11	12c	1 4 4 4 5 5 5 5 5 5 5 5	17e	1 2 2 3 3 3 3 3 4 4 4 5 5 7 10 13 13
8l	2 6 6 7 11 11 11 11	12d	1 3 3 3 4 4 5 6 6 6 6 6	18a	1 2 2 2 3 3 3 3 3 3 4 4 5 6 8 12 12 12
8m	2 6 7 8 12 13 13 13	12e	1 2 3 3 4 4 4 5 7 7 7 7	18b	1 3 3 3 3 3 3 4 4 4 5 5 6 7 11 12 12 12
9a	2 4 4 4 4 4 4 4 4	12f	1 3 4 4 5 5 6 7 7 7 7 7	18c	1 3 3 4 4 4 4 4 5 5 6 6 7 9 12 12 12 12
9b	3 5 5 5 5 5 5 5 5	12g	1 3 4 4 5 6 6 8 8 8 8 8	18d	1 1 2 2 2 2 2 2 3 3 3 3 3 4 5 7 13 13
9c	2 6 6 6 6 6 6 6 6	12h	1 3 4 4 5 6 6 7 9 9 9 9	19a	1 2 2 2 2 2 2 2 3 3 3 4 4 4 5 5 9 12 12
9d	1 4 4 5 6 7 7 7 7	12i	1 2 2 3 3 3 4 5 7 10 10 10	19b	1 2 2 3 3 3 3 3 3 3 4 4 5 5 7 9 12 12 12
9e	2 5 6 7 7 7 7 7 7	12j	1 3 3 3 4 4 5 6 8 10 10 10	19c	1 2 3 3 3 3 3 3 4 4 4 4 5 6 7 10 12 12 12
9f	1 4 5 5 6 8 8 8 8	12k	1 4 4 4 5 5 6 7 10 10 10 10	21e	1 2 2 2 2 2 3 3 3 3 3 3 4 4 4 5 6 9 12 12 12
9g	2 5 5 7 8 8 8 8 8	12l	2 4 4 5 6 7 7 9 10 10 10 10		

PRECAST REINFORCED CONCRETE LINTELS IN CONCRETE BLOCK WALLS

Lintels are tabulated per 4 in. thickness of wall; for 8 in. thickness double the width and reinforcement, and for a 12 in. wall, triple it. Lintels given have a capacity to carry only an equilateral triangle of 32 psf masonry with a base of (L + 8 in.) on a clear span of L; no provisions are made for beams, purlins, or other concentrated loads.

LINTELS IN BLOCK WALLS (per 4 in. thickness)

Clear Span L	Total Length $L+$ 1′-4″	Min. f'_c (psi)	Height of Lintel (in.)	Reinforcement Bottom	Reinforcement Top	Weight of Lintel (lb)
4′-0	5′-4	2000	7⅝	1-#3	1-#2	170
5′-0	6′-4		7⅝	1-#3	1-#2	203
6′-0	7′-4		7⅝	1-#3	1-#2	235
7′-0	8′-4	2500	7⅝	1-#4	1-#2	266
8′-0	9′-4		7⅝	1-#4	1-#2	299
9′-0	10′-4		7⅝	1-#6	1-#2	331
10′-0	11′-4	2000	11⅝	1-#5	1-#2	544
11′-0	12′-4		11⅝	1-#5	1-#2	592
12′-0	13′-4		11⅝	1-#6	1-#2	640
13′-0	14′-4	2500	11⅝	1-#8	1-#3	688



For wall to arch over opening and put only a triangular load on the lintel, there must be:— (1) an unbroken, solid wall above the vertex of the triangle equal to about one-third the height of the triangle, and (2) no included concentrations of load.

Bottom bars only may be used if the lintel is plainly marked, properly handled, and always kept right side up. If plainly marked, but likely to be stressed by drooping in handling, use top bars for transportation purposes. If lintel is not marked and can be installed upside down, use same size bars in top as are scheduled for bottom.

Check texture of exposed surfaces of lintel to harmonize with exposed blocks. Either light weight or standard concrete may be used.

For uniformly loaded lintels see page 232.

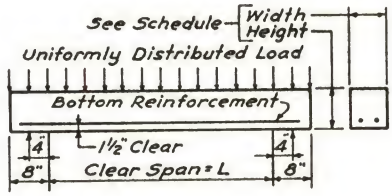
CARRYING CAPACITY OF REINFORCED CONCRETE LINTELS

The table on page 231 gives lintel designs to carry an equilateral triangle of wall. These tables give the safe superimposed uniformly distributed load per lineal foot on 8 x 8 and 12 x 12 in. lintels on spans of 4′-0″ to 12′-0″.

8 x 8 in. Nominal Lintels (7⅝ x 7⅝)

Clear Opening L	Reinforcement					Length of Lintel	Weight of Lintel (lb)
	2-#3	#3 + #4	2-#4	#4 + #5	2-#5		
4′-0	697	1010	1320	1545	1870	5′-4″	340
5′-0	452	662	870	1130	1386	6′-4″	405
6′-0	309	460	612	800	980	7′-4″	469
7′-0	218	330	446	586	730	8′-4″	533
8′-0	157	247	336	446	556	9′-4″	597
9′-0	93	186	258	346	436	10′-4″	661
10′-0	78	136	192	264	338	11′-4″	725
11′-0	58	108	157	219	280	12′-4″	790
12′-0	40	82	124	176	228	13′-4″	854

To left of heavy line, f'_c = 2500 or 3000 psi but to right of heavy line $f'_c \gtrless$ 3000 psi.



Concentrated loads can be approximated by the equivalent uniform loads on page 67, which are fairly accurate for flexure but should be investigated for shear.

The reinforcement *must* be placed in the bottom of the lintel, and can be the only reinforcement if lintel is plainly marked, carefully handled, and always kept right side up; otherwise see notes on page 231.

8 wide x 12 in. high Nominal Lintels (7⅝ x 11⅝)

Clear Opening L	Reinforcement						Length of Lintel	Weight of Lintel (lb)
	2-#4	#4 + #5	2-#5	#5 + #6	2-#6	#6 + #7		
4′-0″	2140	3020	3020	3020	3020	3020	5′-4″	513
5′-0″	1420	1830	2400	2400	2400	2400	6′-4″	609
6′-0″	995	1295	1445	1980	1980	1980	7′-4″	704
7′-0″	725	950	1065	1450	1690	1690	8′-4″	800
8′-0″	545	725	815	1115	1325	1460	9′-4″	896
9′-0″	420	565	635	880	1045	1255	10′-4″	992
10′-0″	315	430	490	680	815	980	11′-4″	1088
11′-0″	260	390	405	575	690	830	12′-4″	1184
12′-0″	205	290	330	470	570	690	13′-4″	1280

Below and to left of heavy line, f'_c = 2500 or 3000 psi, but above and to right of heavy line, $f'_c \gtrless$ 3000 psi. Either light weight or standard concrete may be used.

The above tables give uniformly distributed safe superimposed load per lineal foot; no allowance for beams, purlins, or other concentrations.

AXIALLY LOADED SQUARE TIED CONCRETE COLUMNS

These tables give the safe carrying capacity in kips (1000-lb units) for concentrically loaded square tied reinforced concrete columns of ordinary length (ratio of unsupported height of column to least side, H/t, equal to or less than 10), for concretes of 3000 *, 3750 * and 5000 * psi ultimate strength, with vertical bars of intermediate grade steel (yield point = 40,000 psi) or hard grade steel (yield point = 50,000 psi), computed in accordance with the "Building Code Requirements for Reinforced Concrete (ACI 318-56)." For columns eccentrically loaded, see tables on pages 281 to 360, inclusive.

Tables are given on page 274 for the safe concentric load in kips on steel pipe columns of standard, extra heavy and double extra heavy steel pipe.

The user is referred to the "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)" for much helpful information about exact details of columns and the arrangement of reinforcing bars.

Whenever the ratio of H/t exceeds 10, the tabulated capacity is to be reduced according to the ACI formula, $P' = P\ (1.3\text{-}0.03\ H/t)$. The useable percentage of P for any desired H/t may be taken from the following table:—

H/t	(1.3-0.03 H/t)	H/t	(1.3-0.03 H/t)	
11	0.97	20	0.70	Maximum for columns eccentrically loaded
12	0.94	21	0.67	
13	0.91	22	0.64	
14	0.88	23	0.61	
15	0.85	24	0.58	
16	0.82	25	0.55	
17	0.79	26	0.52	
18	0.76	27	0.49	
19	0.73			

Where lapped splices are used for vertical steel, the bars shall extend above the construction joint (usually the top of the structural slab) 20 nominal bar diameters for intermediate grade, rail or hard grade bars with deformations meeting ASTM A305, as evaluated in the following table:—

Bar Size:—	#5	#6	#7	#8	#9	#10	#11
Lap:—	1′-0½	1′-3	1′-5½	1′-8	1′-10½	2′-1½	2′-4½

* Three thousand psi concrete is usually readily available either ready mixed or job mixed. The use of 3750 psi concrete ordinarily requires a little more care and control and should be used only when job conditions are known to produce this strength. Although 5000 psi concrete can be produced consistently, its use should be restricted to those cases where a program of unusually careful proportioning, control and testing is available to guarantee at least this breaking strength.

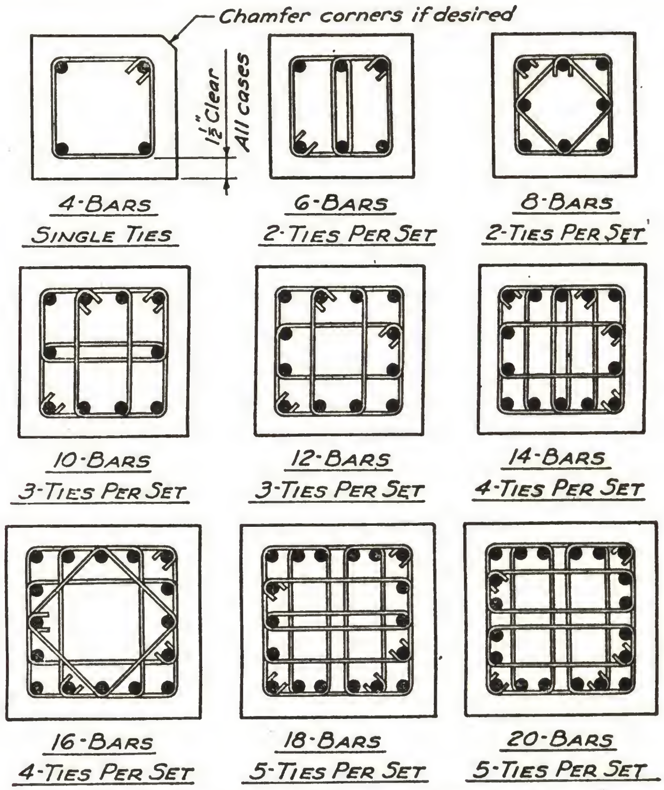
AXIALLY LOADED SQUARE TIED CONCRETE COLUMNS

In lapped splices, column verticals may be offset just below the construction joint at a slope of 1 in. horizontal to 6 in. vertical to come inside of and in contact with the verticals above and ties may be added to care for the stresses developed. Where the offset would exceed about 4 in., separate dowels should be used. [See ACI "Manual of Standard Practice for Detailing Reinforced Concrete Structures," page 13.] [See ACI Code 1103(*c*)4.]

Tie spacing is tabulated as determined by the reinforcement but *must never exceed the least side of the column.* Allow 1½ in. minimum protection over the outside of the ties. Arrange ties as shown in the figure below. Number of ties per set, bar size and spacing between sets of ties are given in the Safe Load Tables. The maximum spacing of column ties is given in the table on page 235.

For a table giving the volumes of concrete in round columns, capitals and square columns, see page 106.

The axial capacity of a round tied column can be approximated by taking 82% * of the capacity of a square tied column with side equal to the diameter



If access to the interior of a column is necessary, a different pattern of ties may be substituted, provided ties are so designed that each vertical bar is securely braced against movement in any direction.

* This varies from 81.2% when A_s = 0.01 of the square column to 88.8% when A_s = 0.04 of the round column, taking into account all grades of steel and concrete.

AXIALLY LOADED SQUARE TIED CONCRETE COLUMNS

MAXIMUM SPACING OF COLUMN TIES

Vertical Bar Size	Size and Spacing of Ties in Inches Maximum Spacing Not to Exceed Least Column Dimension		
	#2	#3	#4
#5	10	10	10
#6	12	12	12
#7	12	14	14
#8	12 *	16	16
#9	12 *	18	18
#10	12 *	18	20
#11	12 *	18	22

* #2 ties are not recommended for #8 or larger verticals.

and with the same number of vertical bars. The axial capacity of a rectangular tied column is fairly close to that of a square column whose side is the mean of the two sides of the rectangle and with the same amount of vertical steel. The minimum side dimension of a main column of reinforced concrete should not be less than 8 in. nor the area less than 120 sq in.

While the scope of these tables is adequate for most purposes, it is not practicable to present all possible combinations of concrete and steel here. For those who want to design a column outside the range of these tables and for those who wish to know how they are computed, the following example will be instructive:—

Example—For the table on page 237, determine the safe axial carrying capacity of a column 35 in. square of 3000 psi concrete reinforced with 18-#10 bars, intermediate grade.

$$P = 0.80\,A_g\,(0.225 f'_c + f_s p_g) = 540\,A_g + 12{,}800\,A_s \ ^*$$

$540 \times 35 \times 35 = 661.5$ kips (Line 2, last column)
$12{,}800 \times 18 \times 1.27 = 292.6$ kips (Column 2, last line)
$P = 954.1$ kips † (Last line, last column)

Ties—Spacing $\lesseqgtr$ 16 bar dia. $\lesseqgtr 16 \times 1.27 \lesseqgtr 20$ in.
$\lesseqgtr$ column side $\lesseqgtr$ 35 in.
$\lesseqgtr$ 48 tie dia. $\lesseqgtr \begin{cases} 48 \times \frac{3}{8} \lesseqgtr 18 \text{ in. if \#3 ties} \\ 48 \times \frac{1}{4} \lesseqgtr 12 \text{ in. if \#2 ties} \end{cases}$

so use 5 sets of #3 @ 18 (Last line, third and fourth columns)

Example—If the column in the previous example were 46′-8″ in unsupported height, what would its safe load be?

$$\frac{H}{t} = \frac{46.67 \times 12}{35} = 16$$

$$P = 954.1\,(1.3 - 0.03 \times 16)$$
$$= 954.1 \times 0.82 \ddagger = 782 \text{ kips}$$

* For nomenclature, see pages 20-21.

† To be on the side of safety, decimal fractions are always dropped in these column tables and never rounded upward.

‡ This value can also be taken from the table on page 233.

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 540\,A_g + 12{,}800\,A_s$$

Vert Bars	12,800 A_s (kips)	Ties: Quant* and Size	Ties: Spcg (in.)	11	12	13	14	15	16	17	18	19	20	21
			540 A_g	65.3	77.7	91.2	105.8	121.5	138.2	156.0	174.9	194.9	216.0	238.1
4-#5	15.8	1-#2	10	81										
4-#6	22.5	1-#2	12	87	100	113								
4-#7	30.7	1-#2	12	96	108	121	136	152						
6-#6	33.7	2-#2	12	99	111	124	139	155	171					
4-#8	40.4	1-#3	16	105	118	131	146	161	178	196				
8-#6	45.0	2-#2	12	110	122	136	150	166	183	201	219			
4-#9	51.2	1-#3	18	116	128	142	157	172	189	207	226	246	267	
6-#8	60.6	2-#3	16	125	138	151	166	182	198	216	235	255	276	298
4-#10	65.0	1-#3	18		142	156	170	186	203	221	239	259	281	303
6-#9	76.8	2-#3	18			168	182	198	215	232	251	271	292	314
4-#11	79.8	1-#3	18			171	185	201	218	235	254	274	295	317
6-#10	97.5	2-#3	18				203	219	235	253	272	292	313	335
8-#9	102.4	2-#3	18					223	240	258	277	297	318	340
6-#11	119.8	2-#3	18						258	275	294	314	335	357
10-#9	128.0	3-#3	18						266	284	302	322	344	366
12-#9	153.6	3-#3	18								328	348	369	391
8-#11	159.7	2-#3	18								334	354	375	397
14-#9	179.2	4-#3	18									374	395	417
12-#10	195.0	3-#3	18										411	433
10-#11	199.6	3-#3	18										415	437
14-#10	227.5	4-#3	18											
12-#11	239.6	3-#3	18											
16-#10	260.0	4-#3	18											
14-#11	279.5	4-#3	18											
18-#10	292.6	5-#3	18											

Column Side (in.): 11–21.

Tie spacing shown is determined by reinforcement but must never exceed the least dimension of the column.

* Quantity refers to number of ties per set (see fig. on page 234).

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$P = 540\ A_g + 12{,}800\ A_s$

22	23	24	25	26	27	28	29	30	31	32	33	34	35
261.3	285.6	311.0	337.5	365.0	393.6	423.3	454.1	486.0	518.9	552.9	588.0	624.2	661.5
326													
338	362	387											
341	365	390											
358	383	408	435	462	491								
363	388	413	439	467	496	525							
381	405	430	457	484	513	543	573	605					
389	413	439	465	493	521	551	582	614	646				1%
414	439	464	491	518	547	576	607	639	672	706	741	777	
421	445	470	497	524	553	583	613	645	678	712	747	783	821
440	464	490	516	544	572	602	633	665	698	732	767	803	840
456	480	506	532	560	588	618	649	681	713	747	783	819	856
460	485	510	537	564	593	622	653	685	718	752	787	823	861
488	513	538	565	592	621	650	681	713	746	780	815	851	889
500	525	550	577	604	633	662	693	725	758	792	827	863	901
	545	571	597	625	653	683	714	746	778	812	848	884	921
	4%	590	617	644	673	702	733	765	798	832	867	903	941
		603	630	657	686	715	746	778	811	845	880	916	954

CONCRETE
$f'_c = 3{,}000$ psi
INT. GRADE VERT. BARS

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 675\ A_g + 12,800\ A_s$$

Vert Bars	12,800 A_s (kips)	Ties: Quant* and Size	Ties: Spcg (in.)	11	12	13	14	15	16	17	18	19	20	21
			Column Side (in.)	11	12	13	14	15	16	17	18	19	20	21
			675 A_g	81.6	97.2	114.0	132.3	151.8	172.8	195.0	218.7	243.6	270.0	297.6
4-#5	15.8	1-#2	10	97										
4-#6	22.5	1-#2	12	104	119	136								
4-#7	30.7	1-#2	12	112	127	144	163	182						
6-#6	33.7	2-#2	12	115	130	147	166	185	206					
4-#8	40.4	1-#3	16	122	137	154	172	192	213	235				
8-#6	45.0	2-#2	12	126	142	159	177	196	217	240	263			
4-#9	51.2	1-#3	18	132	148	165	183	203	224	246	269	294	321	
6-#8	60.6	2-#3	16	142	157	174	192	212	233	255	279	304	330	358
4-#10	65.0	1-#3	18		162	179	197	216	237	260	283	308	335	362
6-#9	76.8	2-#3	18			190	209	228	249	271	295	320	346	374
4-#11	79.8	1-#3	18			193	212	231	252	274	298	323	349	377
6-#10	97.5	2-#3	18				229	249	270	292	316	341	367	395
8-#9	102.4	2-#3	18					254	275	297	321	346	372	400
6-#11	119.8	2-#3	18						292	314	338	363	389	417
10-#9	128.0	3-#3	18						300	323	346	371	398	425
12-#9	153.6	3-#3	18								372	397	423	451
8-#11	159.7	2-#3	18								378	403	429	457
14-#9	179.2	4-#3	18									422	449	476
12-#10	195.0	3-#3	18										465	492
10-#11	199.6	3-#3	18										469	497
14-#10	227.5	4-#3	18											
12-#11	239.6	3-#3	18											
16-#10	260.0	4-#3	18											
14-#11	279.5	4-#3	18											
18-#10	292.6	5-#3	18											

Tie spacing shown is determined by reinforcement but must never exceed the least dimension of the column.

* Quantity refers to number of ties per set (see fig. on page 234).

SQUARE TIED COLUMNS—Safe Axial Load in Kips
$P = 675\,A_g + 12{,}800\,A_s$

CONCRETE
$f'_c = 3{,}750$ psi
INT. GRADE VERT. BARS

22	23	24	25	26	27	28	29	30	31	32	33	34	35
326.7	357.0	388.8	421.8	456.3	492.0	529.2	567.6	607.5	648.6	691.2	735.0	780.3	826.8
391													
403	433	465											
406	436	468											
424	454	486	519	553	589								
429	459	491	524	558	594	631							
446	476	508	541	576	611	649	687	727					
454	485	516	549	584	620	657	695	735	776				
480	510	542	575	609	645	682	721	761	802	844	888	933	1%
486	516	548	581	616	651	688	727	767	808	850	894	940	986
505	536	568	601	635	671	708	746	786	827	870	914	959	1006
521	552	583	616	651	687	724	762	802	843	886	930	975	1021
526	556	588	621	655	691	728	767	807	848	890	934	979	1026
554	584	616	649	683	719	756	795	835	876	918	962	1007	1054
566	596	628	661	695	731	768	807	847	888	930	974	1019	1066
	617	648	681	716	752	789	827	867	908	951	995	1040	1086
	4%	668	701	735	771	808	847	887	928	970	1014	1059	1106
		681	714	748	784	821	860	900	941	983	1027	1072	1119

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 900\,A_g + 12{,}800\,A_s$$

			Column Side (in.)	11	12	13	14	15	16	17	18	19	20	21
			$900\,A_g$	108.9	129.6	152.1	176.4	202.5	230.4	260.1	291.6	324.9	360.0	396.9
Vert Bars	12,800 A_s (kips)	Ties: Quant * and Size	Ties: Spcg (in.)											
4-#5	15.8	1-#2	10	124										
4-#6	22.5	1-#2	12	131	152	174								
4-#7	30.7	1-#2	12	139	160	182	207	233						
6-#6	33.7	2-#2	12	142	163	185	210	236	264					
4-#8	40.4	1-#3	16	149	170	192	216	242	270	300				
8-#6	45.0	2-#2	12	153	174	197	221	247	275	305	336			
4-#9	51.2	1-#3	18	160	180	203	227	253	281	311	342	376	411	
6-#8	60.6	2-#3	16	169	190	212	237	263	291	320	352	385	420	457
4-#10	65.0	1-#3	18		194	217	241	267	295	325	356	389	425	461
6-#9	76.8	2-#3	18			228	253	279	307	336	368	401	436	473
4-#11	79.8	1-#3	18			231	256	282	310	339	371	404	439	476
6-#10	97.5	2-#3	18				273	300	327	357	389	422	457	494
8-#9	102.4	2-#3	18					304	332	362	394	427	462	499
6-#11	119.8	2-#3	18						350	379	411	444	479	516
10-#9	128.0	3-#3	18						358	388	419	452	488	524
12-#9	153.6	3-#3	18								445	478	513	550
8-#11	159.7	2-#3	18								451	484	519	556
14-#9	179.2	4-#3	18									504	539	576
12-#10	195.0	3-#3	18										555	591
10-#11	199.6	3-#3	18										559	596
14-#10	227.5	4-#3	18											
12-#11	239.6	3-#3	18											
16-#10	260.0	4-#3	18											
14-#11	279.5	4-#3	18											
18-#10	292.6	5-#3	18											

Tie spacing shown is determined by reinforcement but must never exceed the least dimension of the column.

* Quantity refers to number of ties per set (see fig. on page 234).

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 900\,A_g + 12{,}800\,A_s$$

22	23	24	25	26	27	28	29	30	31	32	33	34	35
435.6	476.1	518.4	562.5	608.4	656.1	705.6	756.9	810.0	864.9	921.6	980.1	1040.4	1102.5
500													
512	552	595											
515	555	598											
533	573	615	660	705	753								
538	578	620	664	710	758	808							
555	595	638	682	728	775	825	876	929					
563	604	646	690	736	784	833	884	938	992				
589	629	672	716	762	809	859	910	963	1018	1075	1133	1194	1%
595	635	678	722	768	815	865	916	969	1024	1081	1139	1200	1262
614	655	697	741	787	835	884	936	989	1044	1100	1159	1219	1281
630	671	713	757	803	851	900	951	1005	1059	1116	1175	1235	1297
635	675	718	762	808	855	905	956	1009	1064	1121	1179	1240	1302
663	703	745	790	835	883	933	984	1037	1092	1149	1207	1267	1330
675	715	758	802	848	895	945	996	1049	1104	1161	1219	1280	1342
	736	778	822	868	916	965	1016	1070	1124	1181	1240	1300	1362
	4%	797	842	887	935	985	1036	1089	1144	1201	1259	1319	1382
		811	855	901	948	998	1049	1102	1157	1214	1272	1333	1395

CONCRETE
$f'_c = 5{,}000$ psi
INT. GRADE VERT. BARS

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 675\,A_g + 16{,}000\,A_s$$

Vert Bars	16,000 A_s (kips)	Ties: Quant * and Size	Ties: Spcg (in.)	11	12	13	14	15	16	17	18	19	20	21
			Column Side (in.)	11	12	13	14	15	16	17	18	19	20	21
			675 A_g	81.6	97.2	114.0	132.3	151.8	172.8	195.0	218.7	243.6	270.0	297.6
4-#5	19.8	1-#2	10	101										
4-#6	28.1	1-#2	12	109	125	142								
4-#7	38.4	1-#2	12	120	135	152	170	190						
6-#6	42.2	2-#2	12	123	139	156	174	194	215					
4-#8	50.5	1-#3	16	132	147	164	182	202	223	245				
8-#6	56.3	2-#2	12	137	153	170	188	208	229	251	275			
4-#9	64.0	1-#3	18	145	161	178	196	215	236	259	282	307	334	
6-#8	75.8	2-#3	16	157	173	189	208	227	248	270	294	319	345	373
4-#10	81.2	1-#3	18		178	195	213	233	254	276	299	324	351	378
6-#9	96.0	2-#3	18			210	228	247	268	291	314	339	366	393
4-#11	99.8	1-#3	18			213	232	251	272	294	318	343	369	397
6-#10	121.9	2-#3	18				254	273	294	316	340	365	391	419
8-#9	128.0	2-#3	18					279	300	323	346	371	398	425
6-#11	149.7	2-#3	18						322	344	368	393	419	447
10-#9	160.0	3-#3	18						332	355	378	403	430	457
12-#9	192.0	3-#3	18								410	435	462	489
8-#11	199.6	2-#3	18								418	443	469	497
14-#9	224.0	4-#3	18									467	494	521
12-#10	243.8	3-#3	18										513	541
10-#11	249.6	3-#3	18										519	547
14-#10	284.4	4-#3	18											
12-#11	299.5	3-#3	18											
16-#10	325.1	4-#3	18											
14-#11	349.4	4-#3	18											
18-#10	365.7	5-#3	18											

Tie spacing shown is determined by reinforcement but must never exceed the least dimension of the column.

* Quantity refers to number of ties per set (see fig. on page 234).

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 675\,A_g + 16{,}000\,A_s$$

22	23	24	25	26	27	28	29	30	31	32	33	34	35
326.7	357.0	388.8	421.8	456.3	492.0	529.2	567.6	607.5	648.6	691.2	735.0	780.3	826.8
407													
422	453	484											
426	456	488											
448	478	510	543	578	613								
454	485	516	549	584	620	657							
476	506	538	571	606	641	678	717	757					
486	517	548	581	616	652	689	727	767	808				
518	549	580	613	648	684	721	759	799	840	883	927	972	1%
526	556	588	621	655	691	728	767	807	848	890	934	979	1026
550	581	612	645	680	716	753	791	831	872	915	959	1004	1050
570	600	632	665	700	735	773	811	851	892	935	978	1024	1070
576	606	638	671	705	741	778	817	857	898	940	984	1029	1076
611	641	673	706	740	776	813	852	891	933	975	1019	1064	1111
626	656	688	721	755	791	828	867	907	948	990	1034	1079	1126
	682	713	746	781	817	854	892	932	973	1016	1060	1105	1151
	4%	738	771	805	841	878	917	956	998	1040	1084	1129	1176
		754	787	822	857	894	933	973	1014	1056	1100	1146	1192

CONCRETE
$f'_c = 3{,}750$ psi
HARD GRADE VERT. BARS

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 900\ A_g + 16{,}000\ A_s$$

				11	12	13	14	15	16	17	18	19	20	21
			Column Side (in.)	11	12	13	14	15	16	17	18	19	20	21
			$900\ A_g$	108.9	129.6	152.1	176.4	202.5	230.4	260.1	291.6	324.9	360.0	396.9
Vert Bars	16,000 A_s (kips)	Ties: Quant * and Size	Ties: Spcg (in.)											
4-#5	19.8	1-#2	10	128										
4-#6	28.1	1-#2	12	137	157	180								
4-#7	38.4	1-#2	12	147	168	190	214	240						
6-#6	42.2	2-#2	12	151	171	194	218	244	272					
4-#8	50.5	1-#3	16	159	180	202	226	253	280	310				
8-#6	56.3	2-#2	12	165	185	208	232	258	286	316	347			
4-#9	64.0	1-#3	18	172	193	216	240	266	294	324	355	388	424	
6-#8	75.8	2-#3	16	184	205	227	252	278	306	335	367	400	435	472
4-#10	81.2	1-#3	18		210	233	257	283	311	341	372	406	441	478
6-#9	96.0	2-#3	18			248	272	298	326	356	387	420	456	492
4-#11	99.8	1-#3	18			251	276	302	330	359	391	424	459	496
6-#10	121.9	2-#3	18				298	324	352	382	413	446	481	518
8-#9	128.0	2-#3	18					330	358	388	419	452	488	524
6-#11	149.7	2-#3	18						380	409	441	474	509	546
10-#9	160.0	3-#3	18						390	420	451	484	520	556
12-#9	192.0	3-#3	18								483	516	552	588
8-#11	199.6	2-#3	18								491	524	559	596
14-#9	224.0	4-#3	18									548	584	620
12-#10	243.8	3-#3	18										603	640
10-#11	249.6	3-#3	18										609	646
14-#10	284.4	4-#3	18											
12-#11	299.5	3-#3	18											
16-#10	325.1	4-#3	18											
14-#11	349.4	4-#3	18											
18-#10	365.7	5-#3	18											

Tie spacing shown is determined by reinforcement but must never exceed the least dimension of the column.

* Quantity refers to number of ties per set (see fig. on page 234).

SQUARE TIED COLUMNS—Safe Axial Load in Kips

$$P = 900\,A_g + 16{,}000\,A_s$$

CONCRETE
$f'_c = 5{,}000$ psi
HARD GRADE VERT. BARS

22	23	24	25	26	27	28	29	30	31	32	33	34	35
435.6	476.1	518.4	562.5	608.4	656.1	705.6	756.9	810.0	864.9	921.6	980.1	1040.4	1102.5
516													
531	572	614											
535	575	618											
557	598	640	684	730	778								
563	604	646	690	736	784	833							
585	625	668	712	758	805	855	906	959					
595	636	678	722	768	816	865	916	970	1024				
627	668	710	754	800	848	897	948	1002	1056	1113	1172	1232	1%
635	675	718	762	808	855	905	956	1009	1064	1121	1179	1240	1302
659	700	742	786	832	880	929	980	1034	1088	1145	1204	1264	1326
679	719	762	806	852	899	949	1000	1053	1108	1165	1223	1284	1346
685	725	768	812	858	905	955	1006	1059	1114	1171	1229	1290	1352
720	760	802	846	892	940	990	1041	1094	1149	1206	1264	1324	1386
735	775	817	862	907	955	1005	1056	1109	1164	1221	1279	1339	1402
	801	843	887	933	981	1030	1082	1135	1190	1246	1305	1365	1427
	4%	867	911	957	1005	1055	1106	1159	1214	1271	1329	1389	1451
		884	928	974	1021	1071	1122	1175	1230	1287	1345	1406	1468

AXIALLY LOADED SPIRALLY REINFORCED CONCRETE COLUMNS, ROUND OR SQUARE

These tables give the safe carrying capacity in kips (1000-lb units) for concentrically loaded spirally reinforced round or square concrete columns of ordinary length (ratio of unsupported height to least side, H/t, equal to or less than 10), for concretes of 3000 *, 3750 * and 5000 * psi ultimate strength, with vertical bars of intermediate grade steel (yield point = 40,000 psi) or hard grade steel (yield point = 50,000 psi), computed in accordance with the "Building Code Requirements for Reinforced Concrete (ACI 318-56)."

The tables usually give a choice between spirals made of intermediate grade hot rolled rod (yield point = 40,000 psi) or of cold drawn wire (yield point = 60,000 psi). Since the heaviest practicable spiral is $\frac{5}{8}\phi$ @ 2 in. pitch and since some of the larger square columns require more spiral reinforcement than intermediate grade would supply, hard grade spirals of hot rolled rod (yield point = 50,000 psi) are specified in such cases until their capacity is insufficient, after which the tables indicate that cold drawn wire should be used or a double spiral, one inside the other.

For eccentrically loaded spirally reinforced columns, square or round, see pages 281-360.

Designers are cautioned that for prompt delivery only one grade of spiral should be used on any one contract, and, generally, intermediate grade, cold drawn, and hard grade steel are available in the order named.

While these tables are adequate for selecting column sizes from determined loads, the services of a structural engineer are recommended for accuracy in computing loads and designing to avoid eccentricities. The user is referred to the "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)" for much helpful information about exact details of columns and arrangement of reinforcing bars.

Whenever the ratio of H/t exceeds 10, the tabulated capacity is to be reduced according to the ACI formula, $P' = P\,(1.3 - 0.03\,H/t)$. The useable percentage of P for any desired H/t may be taken from the table on page 233.

Where lapped splices are used for vertical steel, the bars shall extend above the horizontal construction joint (usually the top of the structural slab) 20 nominal bar diameters for intermediate grade, rail or hard grade bars with deformations meeting ASTM A305, as evaluated in the table on pages 13 and 233.

* Three thousand psi concrete is usually readily available either ready mixed or job mixed. The use of 3750 psi concrete ordinarily requires a little more care and control and should be used only when job conditions are known to produce this strength. Although 5000 psi concrete can be produced consistently, its use should be restricted to those cases where a program of unusually careful proportioning, control and testing is available to guarantee at least this breaking strength.

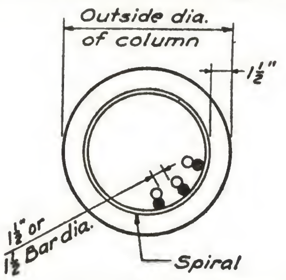
AXIALLY LOADED SPIRALLY REINFORCED CONCRETE COLUMNS

In lapped splices, column verticals may be offset just below the construction joint at a slope of 1 in. horizontal to 6 in. vertical to come inside of and in contact with the verticals above (see page 13). Where the offset would exceed about 4 in., separate dowels should be used. (See "Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)," page 13.)

Spirally reinforced concrete columns (round or square) are to have a minimum clear protection of 1½ in. outside the spiral. Spirals are to have 1½ finishing turns top and bottom and 2 vertical spacers for spirals up to 20 in. outside diameter, 3 for 20 to 30 in. and 4 for 30 in. outside diameter and larger.

The 1956 ACI "Building Code Requirements for Reinforced Concrete" permits up to 8 per cent of vertical bars. These tables are plainly marked

Maximum Number of Column Bars for Columns Carrying Concentric Loads



Vertical bars from below are inside of and in contact with those above.

Outside Diameter of Column	Spiral Size	Bar Size						
		#5	#6	#7	#8	#9	#10	#11
14	⅜φ	12	11	10	9	7	6	—
15	⅜φ	13	12	11	10	8	7	6
16	⅜φ	15	13	12	11	9	8	6
17	⅜φ	16	15	14	12	11	9	7
18	⅜φ	18	16	15	14	12	10	8
19	⅜φ	19	18	16	15	13	11	9
20	⅜φ	21	19	18	16	14	12	10
21	½φ	22	20	19	17	15	13	11
22	½φ	23	22	20	18	16	14	11
23	½φ	25	23	21	20	17	15	12
24	½φ	26	24	22	21	18	16	13
25	½φ	28	26	24	22	19	17	14
26	½φ	29	27	25	23	20	18	15
27	½φ	31	28	26	25	21	19	16
28	⅝φ	32	30	28	26	22	20	17
29	⅝φ	33	31	29	27	23	21	17
30	⅝φ	35	32	30	28	25	22	18
31	⅝φ	36	34	31	29	26	23	19
32	⅝φ	38	35	33	31	27	24	20
33	⅝φ	39	37	34	32	28	25	21
34	⅝φ	41	38	35	33	29	26	22

AXIALLY LOADED SPIRALLY REINFORCED CONCRETE COLUMNS

with a zigzag line above which a clear spacing of 1½ nominal bar diameters or 1 in. minimum can be maintained between the spliced ends of vertical bars from below when carried up inside of and in contact with the verticals above. Larger percentages of vertical steel below the zigzag line require (1) vertical bars larger than the #11 maximum size now in the U. S. Department of Commerce Simplified Practice Recommendation 26 or (2) the butt-welding of vertical bars, one on top of the other, or (3) the use of an additional inner concentric spiral. In lapping vertical bars, no more need be brought up from below than the equivalent of the steel in the column above, arranged in a symmetrical pattern.

Table on page 249 gives the maximum number of various sized verticals that can be accommodated in one ring within different diameters of spirals.

The designer must keep in mind that beam bars often have to extend through the spaces between column verticals. It is generally considered permissible for the horizontal and vertical bars to have point contact in passing, after allowing for the projection of the deformations. Frequently a large-scale layout is helpful in establishing clearances and sometimes a full-size template is used in the field to get the correct orientation of column verticals before the beam bars are installed.

For a table giving the volumes of concrete in round columns, capitals and square columns, see page 106.

While the scope of these tables is adequate for most purposes, it is not practicable to present all possible combinations of concrete and steel here. For those who want to design a column outside the range of these tables and for those who wish to know how they were computed, the following examples will be instructive:—

Example—For the table on page 252, compute the safe axial carrying capacity of a spirally reinforced round concrete column 40 in. in diameter, reinforced with 28-#11 verticals, and proportion the spirals; $f'_c = 3000$ psi, $f_s = 16{,}000$ psi:—

$$P = A_g\,(0.225f'_c + f_s p_g) = 675\,A_g + 16{,}000\,A_s\ *$$

$$\pi \times 20 \times 20 \times 675 = 848.2 \text{ kips (Line 6, last column)}$$

$$16{,}000 \times 28 \times 1.56 = \underline{698.9} \text{ kips (Column 2, last line)}$$

$$P = 1547.1\ \dagger \text{ kips (Last line, last column)}$$

Spirals—$p' = 0.45\left(\dfrac{A_g}{A_c} - 1\right)\dfrac{f'_c}{f'_s}$

$$p' = 0.45\left(\frac{\pi 20 \times 20}{\pi 18.5 \times 18.5} - 1\right)\frac{f'_c}{f'_s}$$

$$= 0.076\frac{f'_c}{f'_s}$$

Hot Rolled Intermediate Grade ($f'_s = 40{,}000$):—$p' = 0.076\dfrac{3{,}000}{40{,}000} = 0.0057 =$ 0.57%

37 in. core ⅜ϕ @ 2 in. pitch = 0.59% (Table on page 271)

(ACI Code Limitations:—Not less than ¼ϕ wire; not less than 1⅜ in. clear; not over 3 in. clear; not over one-sixth the core diameter = $^{37}/_{6}$ = 6⅙ in. pitch.)

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* For nomenclature, see pages 20–21.

† To be on the side of safety, decimal fractions are always dropped in these column tables and never rounded upward.

AXIALLY LOADED SPIRALLY REINFORCED CONCRETE COLUMNS

Cold Drawn (f'_s = 60,000):—$p' = 0.076 \dfrac{3{,}000}{60{,}000} = 0.0038 = 0.38\%$

37 in. core $\frac{3}{8}\phi$ @ 3 in. pitch = 0.39% (Table on page 271)

Example—For the table on page 261, compute the safe axial carrying capacity of a spirally reinforced column 40 in. square of 3750 psi concrete with 28-#11 vertical bars, and proportion the spirals; f_s = 16,000 psi:—

$P = A_g\,(0.225\,f'_c + f_s\,p_g) = 843.75\,A_g + 16{,}000\,A_s$

$843.75 \times 40 \times 40$ = 1350.0 kips (Line 6, last column)

$16{,}000 \times 28 \times 1.56$ = 698.8 kips (Column 2, last line)

P = 2048.8 * kips (Last line, last column)

$$\text{Spirals—}p' = 0.45\left(\frac{40 \times 40}{\pi 18.5 \times 18.5} - 1\right)\frac{f'_c}{f'_s}$$

$$p' = 0.219\frac{f'_c}{f'_s}$$

Hot Rolled Intermediate Grade (f'_s = 40,000):—$p' = 0.219 \dfrac{3{,}750}{40{,}000} = 0.0205 = 2.05\%$

Too heavy a spiral (see the table on page 270), so use hard grade rod.

Hot Rolled Hard Grade (f'_s = 50,000):—$p' = 0.219 \dfrac{3{,}750}{50{,}000} = 0.0164 = 1.64\%$

37 in. core $\frac{5}{8}\phi$ @ 2 in. pitch = 1.67% (Table on page 270)

Cold Drawn (f'_s = 60,000):—$p' = 0.219 \dfrac{3{,}750}{60{,}000} = 0.0137 = 1.37\%$

37 in. core $\frac{5}{8}\phi$ @ $2\frac{1}{4}$ in. pitch = 1.49% (Table on page 270)

For those wishing to obtain percentages of core columes represented by various practicable spirals, table on page 270 is provided. Its use is clearly shown in the following:—

Example—For a 48 in. core, determine the percentage of a $\frac{5}{8}\phi$ spiral with a 2 in. pitch:—

$$p' = \frac{\text{Volume of Steel per Vertical Foot}}{\text{Volume of Core per Vertical Foot}} = \frac{\pi D \dfrac{12}{\text{pitch}} A_s}{\dfrac{\pi D^2}{4} 12} \text{(Table on page 270)}$$

$$p' = \frac{4A_s}{D \times \text{pitch}}$$

$$p' = \frac{4 \times 0.31}{48 \times 2} = 1.29\%$$

Weights of spirals may be obtained from the table on page 272, the notes at the bottom of the page being self-explanatory.

Height of Spirals. The spiral shall extend from the floor level in any story or from the top of footing to the level of the lowest horizontal reinforcement in slab or beam above. In a column with a capital, the spiral shall extend to the plane at which the diameter or width of the capital is twice that of the column.

* To be on the side of safety, decimal fractions are always dropped in these column tables and never rounded upward.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 675\ A_g + 16{,}000\ A_s$$

		16	18	20	22	24	26	28	30	32	34	36	38	40
Outside Dia Col (in.)		16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade	Rod dia	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	1¾	1¾	1¾	2	2	2	2	2	2	2	2	2	2
Cold Drawn Spirals	Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	2	2½	2¾	3	3	3	3	3	3	3	3	3	3
	$675\ A_g$ (kips)	135.7	171.8	212.1	256.6	305.4	358.4	415.6	477.1	542.9	612.8	687.1	765.5	848.2
Vert Bars Quant-Size	$16{,}000\ A_s$ (kips)													
6-#6	42.2	177	214											
6-#7	57.6	193	229	269	314									
6-#8	75.8	211	247	287	332	381								
6-#9	96.0	231	267	308	352	401	454							
6-#10	121.9	257	293	334	378	427	480	537	599					
8-#9	128.0	263	299	340	384	433	486	543	605					
6-#11	149.8	285	321	361	406	455	508	565	626	692	762			
8-#10	162.6	298	334	374	419	468	521	578	639	705	775			
8-#11	199.7	335	371	411	456	505	558	615	676	742	812	886	965	1%
10-#11	249.6	385	421	461	506	555	608	665	726	792	862	936	1015	1097
12-#11	299.5		471	511	556	604	657	715	776	842	912	986	1065	1147
14-#11	349.4			561	606	654	707	765	826	892	962	1036	1114	1197
16-#11	399.4			611	656	704	757	815	876	942	1012	1086	1164	1247
18-#11	449.3				705	754	807	864	926	992	1062	1136	1214	1297
20-#11	499.2					804	857	914	976	1042	1112	1186	1264	1347
22-#11	549.1					854	907	964	1026	1092	1161	1236	1314	1397
24-#11	599.1						957	1014	1076	1141	1211	1286	1364	1447
26-#11	649.0						1007	1064	1126	1191	1261	1336	1414	1497
28-#11	698.9						8%	1114	1176	1241	1311	1386	1464	1547

CONCRETE
$f'_c = 3{,}000$ psi
INT. GRADE VERT. BARS

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 843.75\ A_g + 16{,}000\ A_s$$

	Outside Dia Col (in.)	16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade	Rod dia	½	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2½	2¾	2¾	2¾	2¾	2¾	2¾	3	3	3	3	3
Cold Drawn Spirals	Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	2	2¼	2¼	2¼	2¼	2¼	2¼	2¼	2½	2½	2½	2½	2½
	843.75 A_g (kips)	169.6	214.7	265.0	320.7	381.7	447.9	519.5	596.4	678.5	766.0	858.8	956.9	1060.2
Vert Bars Quant-Size	16,000 A_s (kips)													
6-#6	42.2	211	256											
6-#7	57.6	227	272	322	378									
6-#8	75.8	245	290	340	396	457								
6-#9	96.0	265	310	361	416	477	543							
6-#10	121.9	291	336	386	442	503	569	641	718					
8-#9	128.0	297	342	393	448	509	575	647	724					
6-#11	149.8	319	364	414	470	531	597	669	746	828	915			
8-#10	162.6	332	377	427	483	544	610	682	759	841	928			
8-#11	199.7	369	414	464	520	581	647	719	796	878	965	1058	1156	1%
10-#11	249.6	419	464	514	570	631	697	769	846	928	1015	1108	1206	1309
12-#11	299.5		514	564	620	681	747	819	895	978	1065	1158	1256	1359
14-#11	349.4			614	670	731	797	868	945	1027	1115	1208	1306	1409
16-#11	399.4			664	720	781	847	918	995	1077	1165	1258	1356	1459
18-#11	449.3				770	831	897	968	1045	1127	1215	1308	1406	1509
20-#11	499.2					880	947	1018	1095	1177	1265	1358	1456	1559
22-#11	549.1					930	997	1068	1145	1227	1315	1407	1506	1609
24-#11	599.0						1046	1118	1195	1277	1365	1457	1555	1659
26-#11	649.0						1096	1168	1245	1327	1415	1507	1605	1709
28-#11	698.9					8%		1218	1295	1377	1464	1557	1655	1759

CONCRETE
$f'_c = 3{,}750$ psi
INT. GRADE VERT. BARS

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 1125\,A_g + 16{,}000\,A_s$$

Outside Dia Col (in.)		16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade	Rod dia	½	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2	2	2	2	2	2	2¼	2¼	2¼	2¼	2¼	2¼
Cold Drawn Spirals	Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾
$1125\,A_g$ (kips)		226.1	286.2	353.4	427.6	508.9	597.2	692.7	795.2	904.7	1021.4	1145.1	1275.8	1413.6
Vert Bars Quant-Size	$16{,}000\,A_s$ (kips)													
6-#6	42.2	268	328											
6-#7	57.6	283	343	411	485									
6-#8	75.8	301	362	429	503	584								
6-#9	96.0	322	382	449	523	604	693							
6-#10	121.9	348	408	475	549	630	719	814	917					
8-#9	128.0	354	414	481	555	636	725	820	923					
6-#11	149.8	375	436	503	577	658	747	842	945	1054	1171			
8-#10	162.6	388	448	516	590	671	759	855	957	1067	1184			
8-#11	199.7	425	485	553	627	708	796	892	994	1104	1221	1344	1475	1%
10-#11	249.6	475	535	603	677	758	847	942	1044	1154	1271	1394	1525	1663
12-#11	299.5		585	652	727	808	896	992	1094	1204	1320	1444	1575	1713
14-#11	349.4			702	777	858	946	1042	1144	1254	1370	1494	1625	1763
16-#11	399.4			752	827	908	996	1092	1194	1304	1420	1544	1675	1813
18-#11	449.3				876	958	1046	1142	1244	1354	1470	1594	1725	1862
20-#11	499.2					1008	1096	1191	1294	1403	1520	1644	1775	1912
22-#11	549.1					1058	1146	1241	1344	1453	1570	1694	1824	1962
24-#11	599.0						1196	1291	1394	1503	1620	1744	1874	2012
26-#11	649.0						1246	1341	1444	1553	1670	1794	1924	2062
28-#11	698.9						8%	1391	1494	1603	1720	1844	1974	2112

CONCRETE
$f'_c = 5{,}000$ psi
INT. GRADE VERT. BARS

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 675\ A_g + 20{,}000\ A_s$$

CONCRETE
$f'_c = 3{,}000$ psi
HARD GRADE VERT. BARS

		Outside Dia Col (in.)	16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade		Rod dia	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
		Pitch	1¾	1¾	1¾	2	2	2	2	2	2	2	2	2	2
Cold Drawn Spirals		Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
		Pitch	2	2½	2¾	3	3	3	3	3	3	3	3	3	3
		$675\ A_g$(kips)	135.7	171.8	212.1	256.6	305.4	358.4	415.6	477.1	542.9	612.8	687.1	765.5	848.2
Vert Bars Quant-Size	$20{,}000\ A_s$ (kips)														
6-#6	52.8		188	224											
6-#7	72.0		207	243	284	328									
6-#8	94.8		230	266	306	351	400								
6-#9	120.0		255	291	332	376	425	478							
6-#10	152.4		288	324	364	409	457	510	568	629					
8-#9	160.0		295	331	372	416	465	518	575	637					
6-#11	187.2		322	359	399	443	492	545	602	664	730	800			
8-#10	203.2		338	375	415	459	508	561	618	680	746	816			
8-#11	249.6		385	421	461	506	555	608	665	726	792	862	936	1015	1%
10-#11	312.0		447	483	524	568	617	670	727	789	854	924	999	1077	1160
12-#11	374.4			546	586	631	679	732	790	851	917	987	1061	1139	1222
14-#11	436.8				648	693	742	795	852	913	979	1049	1123	1202	1285
16-#11	499.2				711	755	804	857	914	976	1042	1112	1186	1264	1347
18-#11	561.6					818	867	920	977	1038	1104	1174	1248	1327	1409
20-#11	624.0						929	982	1039	1101	1166	1236	1311	1389	1472
22-#11	686.4						991	1044	1102	1163	1229	1299	1373	1451	1534
24-#11	748.8							1107	1164	1225	1291	1361	1435	1514	1597
26-#11	811.2							1169	1226	1288	1354	1424	1498	1576	1659
28-#11	873.6						8%		1289	1350	1416	1486	1560	1639	1721

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 843.75\ A_g + 20{,}000\ A_s$$

Outside Dia Col (in.)		16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade	Rod dia	½	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2½	2¾	2¾	2¾	2¾	2¾	2¾	3	3	3	3	3
Cold Drawn Spirals	Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	2	2¼	2¼	2¼	2¼	2¼	2¼	2¼	2½	2½	2½	2½	2½
	843.75 A_g (kips)	169.6	214.7	265.0	320.7	381.7	447.9	519.5	596.4	678.5	766.0	858.8	956.9	1060.2
Vert Bars Quant-Size	20,000 A_s (kips)													
6-#6	52.8	222	267											
6-#7	72.0	241	286	337	392									
6-#8	94.8	264	309	359	415	476								
6-#9	120.0	289	334	385	440	501	567							
6-#10	152.4	322	367	417	473	534	600	671	748					
8-#9	160.0	329	374	425	480	541	607	679	756					
6-#11	187.2	356	401	452	507	568	635	706	783	865	953			
8-#10	203.2	372	417	468	523	584	651	722	799	881	969			
8-#11	249.6	419	464	514	570	631	697	769	846	928	1015	1108	1206	1%
10-#11	312.0	481	526	577	632	693	759	831	908	990	1078	1170	1268	1372
12-#11	374.4		589	639	695	756	822	893	970	1052	1140	1233	1331	1434
14-#11	436.8			701	757	818	884	956	1033	1115	1202	1295	1393	1497
16-#11	499.2			764	819	880	947	1018	1095	1177	1265	1358	1456	1559
18-#11	561.6				882	943	1009	1081	1158	1240	1327	1420	1518	1621
20-#11	624.0					1005	1071	1143	1220	1302	1390	1482	1580	1684
22-#11	686.4					1068	1134	1205	1282	1364	1452	1545	1643	1746
24-#11	748.8						1196	1268	1345	1427	1514	1607	1705	1809
26-#11	811.2						8%	1330	1407	1489	1577	1670	1768	1871
28-#11	873.6							1393	1470	1552	1639	1732	1830	1933

CONCRETE
$f'_c = 3{,}750$ psi
HARD GRADE VERT. BARS

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—Safe Axial Load in Kips

$$P = 1125\ A_g + 20{,}000\ A_s$$

Outside Dia Col (in.)		16	18	20	22	24	26	28	30	32	34	36	38	40
Hot Rolled Spirals Intermediate Grade	Rod dia	½	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2	2	2	2	2	2	2¼	2¼	2¼	2¼	2¼	2¼
Cold Drawn Spirals	Wire	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜	⅜
	Pitch	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾	1¾
1125 A_g (kips)		226.1	286.2	353.4	427.6	508.9	597.2	692.7	795.2	904.7	1021.4	1145.1	1275.8	1413.6
Vert Bars Quant-Size	20,000 A_s (kips)													
6-#6	52.8	278	339											
6-#7	72.0	298	358	425	499									
6-#8	94.8	320	381	448	522	603								
6-#9	120.0	346	406	473	547	628	717							
6-#10	152.4	378	438	505	580	661	749	845	947					
8-#9	160.0	386	446	513	587	668	757	852	955					
6-#11	187.2	413	473	540	614	696	784	879	982	1091	1208			
8-#10	203.2	429	489	556	630	712	800	895	998	1107	1224			
8-#11	249.6	475	535	603	677	758	846	942	1044	1154	1271	1394	1525	1%
10-#11	312.0	538	598	665	739	820	909	1004	1107	1216	1333	1457	1587	1725
12-#11	374.4		660	727	802	883	971	1067	1169	1279	1395	1519	1650	1788
14-#11	436.8			790	864	945	1034	1129	1232	1341	1458	1581	1712	1850
16-#11	499.2			852	926	1008	1096	1191	1294	1403	1520	1644	1775	1912
18-#11	561.6				989	1070	1158	1254	1356	1466	1583	1706	1837	1975
20-#11	624.0					1132	1221	1316	1419	1528	1645	1769	1899	2037
22-#11	686.4					1195	1283	1379	1481	1591	1707	1831	1962	2100
24-#11	748.8						1346	1441	1544	1653	1770	1893	2024	2162
26-#11	811.2						1408	1503	1606	1715	1832	1956	2087	2224
28-#11	873.6					8%		1566	1668	1778	1895	2018	2149	2287

CONCRETE
$f'_c = 5{,}000$ psi
HARD GRADE VERT. BARS

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 675\,A_g + 16{,}000\,A_s$$

Side of Column (in.)		16	17	18	19	20	21	22	23	24	25	26	27
Hot Rolled Spirals Intermediate Grade	Rod dia	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
	Pitch	2	2¼	2½	2½	2¾	2¾	2½	2½	2½	2½	2½	2½
Cold Drawn Spirals	Wire	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2¼	2½	2½	2½	2½	2½	2½	2½	2½	2¼	2¼
$675\,A_g$ (kips)		172.8	195.0	218.7	243.6	270.0	297.6	326.7	357.0	388.8	421.8	456.3	492.0
Vert Bars Quant-Size	$16{,}000\,A_s$ (kips)												
6-#6	42.2	215											
6-#7	57.6	230	252	276									
6-#8	75.8	248	270	294	319	345	373						
6-#9	96.0	268	291	314	339	366	393	422	453	484			
6-#10	121.9	294	316	340	365	391	419	448	478	510	543	578	613
8-#9	128.0	300	323	346	371	398	425	454	485	516	549	584	620
6-#11	149.8	322	344	368	393	419	447	476	506	538	571	606	641
8-#10	162.6	335	357	381	406	432	460	489	519	551	584	618	654
8-#11	199.6	372	394	418	443	469	497	526	556	588	621	655	691
10-#11	249.6	422	444	468	493	519	547	576	606	638	671	705	741
12-#11	299.5	472	494	518	543	569	597	626	656	688	721	755	791
14-#11	349.4		544	568	593	619	647	676	706	738	771	805	841
16-#11	399.4			618	643	669	697	726	756	788	821	855	891
18-#11	449.2				692	719	746	775	806	838	871	905	941
20-#11	499.2					769	796	825	856	888	921	955	991
22-#11	549.1						846	875	906	937	970	1005	1041
24-#11	599.0							925	956	987	1020	1055	1091
26-#11	648.9							8%	1005	1037	1070	1105	1140
28-#11	698.8									1087	1120	1155	1190

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 675\ A_g + 16{,}000\ A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
2¼	2¼	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
½	½	½	½	½	½	½	½	½	½	½	½	5/8
2¼	2¼	2¼	2¼	2	2	2	2	2	2	2	2	3
529.2	567.6	607.5	648.6	691.2	735.0	780.3	826.8	874.8	924.0	974.7	1026.6	1080.0
657												
679	717	757										
691	730	770	811									
728	767	807	848	890	934	979	1026					
778	817	857	898	940	984	1029	1076	1124	1173	1224	1276	1%
828	867	907	948	990	1034	1079	1126	1174	1223	1274	1326	1379
878	917	956	998	1040	1084	1129	1176	1224	1273	1324	1376	1429
928	967	1006	1048	1090	1134	1179	1226	1274	1323	1374	1426	1479
978	1016	1056	1097	1140	1184	1229	1276	1324	1373	1423	1475	1529
1028	1066	1106	1147	1190	1234	1279	1326	1374	1423	1473	1525	1579
1078	1116	1156	1197	1240	1284	1329	1375	1423	1473	1523	1575	1629
1128	1166	1206	1247	1290	1334	1379	1425	1473	1523	1573	1625	1679
1178	1216	1256	1297	1340	1383	1429	1475	1523	1572	1623	1675	1728
1228	1266	1306	1347	1390	1433	1479	1525	1573	1622	1673	1725	1778

CONCRETE
f'_c = 3,000 psi
INT. GRADE VERT. BARS

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 843.75\,A_g + 16{,}000\,A_s$$

Side of Column (in.)		16	17	18	19	20	21	22	23	24	25	26	27
Hot Rolled Spirals Int. Grade *	Rod dia	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
	Pitch	2	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
Hot Rolled Spirals Hard Grade *	Rod dia	Use Intermediate Grade											
	Pitch												
Cold Drawn Spirals *	Wire	½	½	½	½	½	½	½	½	½	½	⅝	⅝
	Pitch	2	2¼	2¼	2	2	2	2	2	2	2	3	3
$843.75\,A_g$ (kips)		216.0	243.8	273.3	304.5	337.5	372.0	408.3	446.3	486.0	527.3	570.3	615.0
Vert Bars Quant-Size	$16{,}000\,A_s$ (kips)												
6-#6	42.2	258											
6-#7	57.6	273	301	330									
6-#8	75.8	291	319	349	380	413	447						
6-#9	96.0	312	339	369	400	433	468	504	542	582			
6-#10	121.9	337	365	395	426	459	493	530	568	607	649	692	736
8-#9	128.0	344	371	401	432	465	500	536	574	614	655	698	743
6-#11	149.8	365	393	423	454	487	521	558	596	635	677	720	764
8-#10	162.6	378	406	435	467	500	534	570	608	648	689	732	777
8-#11	199.6	415	443	472	504	537	571	607	645	685	726	769	814
10-#11	249.6	465	493	522	554	587	621	657	695	735	776	819	864
12-#11	299.5	515	543	572	604	637	671	707	745	785	826	869	914
14-#11	349.4		593	622	653	686	721	757	795	835	876	919	964
16-#11	399.4			672	703	736	771	807	845	885	926	969	1014
18-#11	449.2				753	786	821	857	895	935	976	1019	1064
20-#11	499.2					836	871	907	945	985	1026	1069	1114
22-#11	549.1						921	957	995	1035	1076	1119	1164
24-#11	599.0							1007	1045	1085	1126	1169	1214
26-#11	648.9								1095	1134	1176	1219	1263
28-#11	698.8							8%		1184	1226	1269	1313

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

* To facilitate deliveries and eliminate errors, use only one grade of spiral on any one contract, generally available in the order intermediate grade hot rolled rod, cold drawn wire and hard grade hot rolled rod.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 843.75\ A_g + 16{,}000\ A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
						Use Hard Grade						
⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
2¼	2¼	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
2¾	2¾	2¾	2¾	2¾	2½	2½	2½	2½	2½	2½	2¼	2¼
661.5	709.5	759.3	810.8	864.0	918.8	975.3	1033.5	1093.5	1155.0	1218.3	1283.3	1350.0
							CONCRETE $f'_c = 3{,}750$ psi INT. GRADE VERT. BARS					
789												
811	859	909										
824	872	921	973									
861	909	958	1010	1063	1118	1174	1233					
911	959	1008	1060	1113	1168	1224	1283	1343	1404	1467	1532	1%
961	1009	1058	1110	1163	1218	1274	1333	1393	1454	1517	1582	1649
1010	1058	1108	1160	1213	1268	1324	1382	1442	1504	1567	1632	1699
1060	1108	1158	1210	1263	1318	1374	1432	1492	1554	1617	1682	1749
1110	1158	1208	1260	1313	1368	1424	1482	1542	1604	1667	1732	1799
1160	1208	1258	1310	1363	1418	1474	1532	1592	1654	1717	1782	1849
1210	1258	1308	1359	1413	1467	1524	1582	1642	1704	1767	1832	1899
1260	1308	1358	1409	1463	1517	1574	1632	1692	1754	1817	1882	1949
1310	1358	1408	1459	1512	1567	1624	1682	1742	1803	1867	1932	1998
1360	1408	1458	1509	1562	1617	1674	1732	1792	1853	1917	1982	2048

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 1125\,A_g + 16{,}000\,A_s$$

Side of Column (in.)		16	17	18	19	20	21	22	23	24	25	26	27
Hot Rolled Spirals Hard Grade*	Rod dia	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	Use Cold Drawn Wire			
	Pitch	2	2	2	2	2	2	2	2				
Cold Drawn Spirals*	Wire	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
	Pitch	2	2¼	2½	2½	2½	2½	2¼	2¼	2¼	2¼	2¼	2¼
	$1125\,A_g$ (kips)	288.0	325.1	364.5	406.1	450.0	496.1	544.5	595.1	648.0	703.1	760.5	820.1
Vert Bars Quant-Size	$16{,}000\,A_s$ (kips)												
6-#6	42.2	330											
6-#7	57.6	345	382	422									
6-#8	75.8	363	400	440	481	525	571						
6-#9	96.0	384	421	460	502	546	592	640	691	744			
6-#10	121.9	409	447	486	528	571	618	666	717	769	825	882	942
8-#9	128.0	416	453	492	534	578	624	672	723	776	831	888	948
6-#11	149.8	437	474	514	555	599	645	694	744	797	852	910	969
8-#10	162.6	450	487	527	568	612	658	707	757	810	865	923	982
8-#11	199.6	487	524	564	605	649	695	744	794	847	902	960	1019
10-#11	249.6	537	574	614	655	699	745	794	844	897	952	1010	1069
12-#11	299.5	587	624	664	705	749	795	844	894	947	1002	1060	1119
14-#11	349.4		674	713	755	799	845	893	944	997	1052	1109	1169
16-#11	399.4			763	805	849	895	943	994	1047	1102	1159	1219
18-#11	449.2				855	899	945	993	1044	1097	1152	1209	1269
20-#11	499.2					949	995	1043	1094	1147	1202	1259	1319
22-#11	549.1						1045	1093	1144	1197	1252	1309	1369
24-#11	599.0							1143	1194	1247	1302	1359	1419
26-#11	648.9								1244	1296	1352	1409	1469
28-#11	698.8							8%		1346	1401	1459	1518

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

* To facilitate deliveries and eliminate errors, use only one grade of spiral on any one contract, generally available in the order intermediate grade hot rolled rod, cold drawn wire and hard grade hot rolled rod.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 1125\,A_g + 16{,}000\,A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
Use Cold Drawn Wire												
⅝	⅝	⅝	⅝	⅝	⅝	⅝	Use Double Concentric Spirals					
2	2	2	2	2	2	2	$p'=$ .0196	.0193	.0190	.0188	.0185	.0183
882.0	946.1	1012.5	1081.1	1152.0	1225.1	1300.5	1378.1	1458.0	1540.1	1624.5	1711.1	1800.0
1010												
1031	1095	1162										
1044	1108	1175	1243									
1081	1145	1212	1280	1351	1424	1500	1577					
1131	1195	1262	1330	1401	1474	1550	1627	1707	1789	1874	1960	1%
1181	1245	1312	1380	1451	1524	1600	1677	1757	1839	1924	2010	2099
1231	1295	1361	1430	1501	1574	1649	1727	1807	1889	1973	2060	2149
1281	1345	1411	1480	1551	1624	1699	1777	1857	1939	2023	2110	2199
1331	1395	1461	1530	1601	1674	1749	1827	1907	1989	2073	2160	2249
1381	1445	1511	1580	1651	1724	1799	1877	1957	2039	2123	2210	2299
1431	1495	1561	1630	1701	1774	1849	1927	2007	2089	2173	2260	2349
1481	1545	1611	1680	1751	1824	1899	1977	2057	2139	2223	2310	2399
1530	1595	1661	1730	1800	1874	1949	2027	2106	2189	2273	2360	2448
1580	1644	1711	1779	1850	1923	1999	2076	2156	2238	2323	2409	2498

CONCRETE
$f'_c = 5{,}000$ psi
INT. GRADE VERT. BARS

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 675\ A_g + 20{,}000\ A_s$$

Side of Column (in.)		16	17	18	19	20	21	22	23	24	25	26	27
Hot Rolled Spirals Intermediate Grade*	Rod dia	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
	Pitch	2	2¼	2½	2½	2¾	2¾	2½	2½	2½	2½	2½	2½
Cold Drawn Spirals*	Wire	½	½	½	½	½	½	½	½	½	½	½	½
	Pitch	2	2¼	2½	2½	2½	2½	2½	2½	2½	2½	2¼	2¼
	$675\ A_g$ (kips)	172.8	195.0	218.7	243.6	270.0	297.0	326.7	357.0	388.7	421.8	456.3	492.0
Vert Bars Quant-Size	$20{,}000\ A_s$ (kips)												
6-#6	52.8	225											
6-#7	72.0	244	267	290									
6-#8	94.8	267	289	313	338	364	391						
6-#9	120.0	292	315	338	363	390	417	446	477	508			
6-#10	152.4	325	347	371	396	422	449	479	509	541	574	608	644
8-#9	160.0	332	355	378	403	430	457	486	517	548	581	616	652
6-#11	187.2	360	382	405	430	457	484	513	544	575	609	643	679
8-#10	203.2	376	398	421	446	473	500	529	560	591	625	659	695
8-#11	249.6	422	444	468	493	519	546	576	606	638	671	705	741
10-#11	312.0	484	507	530	555	582	609	638	669	700	733	768	804
12-#11	374.4	547	569	593	618	644	671	701	731	763	796	830	866
14-#11	436.8		631	655	680	706	733	763	793	825	858	893	928
16-#11	499.2			717	742	769	796	825	856	887	921	955	991
18-#11	561.6				805	831	858	888	918	950	983	1017	1053
20-#11	624.0					894	921	950	981	1012	1045	1080	1116
22-#11	686.4						983	1013	1043	1075	1108	1142	1178
24-#11	748.8							1075	1105	1137	1170	1205	1240
26-#11	811.2								1168	1199	1233	1267	1303
28-#11	873.6							8%		1262	1295	1329	1365

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

* To facilitate deliveries and eliminate errors, use only one grade of spiral on any one contract, generally available in the order intermediate grade hot rolled rod, cold drawn wire and hard grade hot rolled rod.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 675\,A_g + 20{,}000\,A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
2¼	2¼	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
½	½	½	½	½	½	½	½	½	½	½	½	5/8
2¼	2¼	2¼	2¼	2	2	2	2	2	2	2	2	3
529.2	567.6	607.5	648.6	691.2	735.0	780.3	826.8	874.8	924.0	974.7	1026.6	1080.0
						CONCRETE $f'_c = 3{,}000$ psi HARD GRADE VERT. BARS						
689												
716	754	794										
732	770	810	851									
778	817	857	898	940	984	1029	1076					
841	879	919	960	1003	1047	1092	1138	1186	1236	1286	1338	1%
903	942	981	1023	1065	1109	1154	1201	1249	1298	1349	1401	1454
966	1004	1044	1085	1128	1171	1217	1263	1311	1360	1411	1463	1516
1028	1066	1106	1147	1190	1234	1279	1326	1374	1423	1473	1525	1579
1090	1129	1169	1210	1252	1296	1341	1388	1436	1485	1536	1588	1641
1153	1191	1231	1272	1315	1359	1404	1450	1498	1548	1598	1650	1704
1215	1254	1293	1335	1377	1421	1466	1513	1561	1610	1661	1713	1766
1278	1316	1356	1397	1440	1483	1529	1575	1623	1672	1723	1775	1828
1340	1378	1418	1459	1502	1546	1591	1638	1686	1735	1785	1837	1891
1402	1441	1481	1522	1564	1608	1653	1700	1748	1797	1848	1900	1953

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 843.75\ A_g + 20{,}000\ A_s$$

		16	17	18	19	20	21	22	23	24	25	26	27
Side of Column (in.)		**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**
Hot Rolled Spirals Int. Grade *	Rod dia	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
	Pitch	2	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
Hot Rolled Spirals Hard Grade *	Rod dia		Use Intermediate Grade										
	Pitch												
Cold Drawn Spirals *	Wire	½	½	½	½	½	½	½	½	½	½	⅝	⅝
	Pitch	2	2¼	2¼	2	2	2	2	2	2	2	3	3
$843.75\ A_g$ (kips)		216.0	243.8	273.3	304.5	337.5	372.0	408.3	446.3	486.0	527.3	570.3	615.0
Vert Bars Quant-Size	$20{,}000\ A_s$ (kips)												
6-#6	52.8	268											
6-#7	72.0	288	315	345									
6-#8	94.8	310	338	368	399	432	466						
6-#9	120.0	336	363	393	424	457	492	528	566	606			
6-#10	152.4	368	396	425	456	489	524	560	598	638	679	722	767
8-#9	160.0	376	403	433	464	497	532	568	606	646	687	730	775
6-#11	187.2	403	431	460	491	524	559	595	633	673	714	757	802
8-#10	203.2	419	447	476	507	540	575	611	649	689	730	773	818
8-#11	249.6	465	493	522	554	587	621	657	695	735	776	819	864
10-#11	312.0	528	555	585	616	649	684	720	758	798	839	882	927
12-#11	374.4	590	618	647	678	711	746	782	820	860	901	944	989
14-#11	436.8		680	710	741	774	809	845	883	922	964	1007	1051
16-#11	499.2			772	803	836	871	907	945	985	1026	1069	1114
18-#11	561.6				866	899	933	969	1007	1047	1088	1131	1176
20-#11	624.0					961	996	1032	1070	1110	1151	1194	1239
22-#11	686.4						1058	1094	1132	1172	1213	1256	1301
24-#11	748.8							1157	1195	1234	1276	1319	1363
26-#11	811.2								1257	1297	1338	1381	1426
28-#11	873.6								8%	1359	1400	1443	1488

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

* To facilitate deliveries and eliminate errors, use only one grade of spiral on any one contract, generally available in the order intermediate grade hot rolled rod, cold drawn wire and hard grade hot rolled rod.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 843.75\ A_g + 20{,}000\ A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
						Use Hard Grade						
5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
2¼	2¼	2¼	2¼	2¼	2¼	2	2	2	2	2	2	2
5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8	5/8
2¾	2¾	2¾	2¾	2¾	2½	2½	2½	2½	2½	2½	2¼	2¼
661.5	709.5	759.3	810.8	864.0	918.8	975.3	1033.5	1093.5	1155.0	1218.3	1283.3	1350.0
821												
848	896	946										
864	912	962	1014									
911	959	1008	1060	1113	1168	1224	1283					
973	1021	1071	1122	1176	1230	1287	1345	1405	1467	1530	1595	1%
1035	1083	1133	1185	1238	1293	1349	1407	1467	1529	1592	1657	1724
1098	1146	1196	1247	1300	1355	1412	1470	1530	1591	1655	1720	1786
1160	1208	1258	1310	1363	1418	1474	1532	1592	1654	1717	1782	1849
1223	1271	1320	1372	1425	1480	1536	1595	1655	1716	1779	1844	1911
1285	1333	1383	1434	1488	1542	1599	1657	1717	1779	1842	1907	1974
1347	1395	1445	1497	1550	1605	1661	1719	1779	1841	1904	1969	2036
1410	1458	1508	1559	1612	1667	1724	1782	1842	1903	1967	2032	2098
1472	1520	1570	1622	1675	1730	1786	1844	1904	1966	2029	2094	2161
1535	1583	1632	1684	1737	1792	1848	1907	1967	2028	2091	2156	2223

CONCRETE
$f'_c = 3{,}750$ psi
HARD GRADE VERT. BARS

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 1125\,A_g + 20{,}000\,A_s$$

Side of Column (in.)		16	17	18	19	20	21	22	23	24	25	26	27
Hot Rolled Spirals Hard Grade*	Rod dia	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	Use Cold Drawn Wire			
	Pitch	2	2	2	2	2	2	2	2				
Cold Drawn Spirals*	Wire	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝	⅝
	Pitch	2	2¼	2½	2½	2½	2½	2¼	2¼	2¼	2¼	2¼	2¼
$1125\,A_g$ (kips)		288.0	325.1	364.5	406.1	450.0	496.1	544.5	595.1	648.0	703.1	760.5	820.1
Vert Bars Quant-Size	$20{,}000\,A_s$ (kips)												
6-#6	52.8	340											
6-#7	72.0	360	397	436									
6-#8	94.8	382	419	459	500	544	590						
6-#9	120.0	408	445	484	526	570	616	664	715	768			
6-#10	152.4	440	477	516	558	602	648	696	747	800	855	912	972
8-#9	160.0	448	485	524	566	610	656	704	755	808	863	920	980
6-#11	187.2	475	512	551	593	637	683	731	782	835	890	947	1007
8-#10	203.2	491	528	567	609	653	699	747	798	851	906	963	1023
8-#11	249.6	537	574	614	655	699	745	794	844	897	952	1010	1069
10-#11	312.0	600	637	676	718	762	808	856	907	960	1015	1072	1132
12-#11	374.4	662	699	738	780	824	870	918	969	1022	1077	1134	1194
14-#11	436.8		761	801	842	886	932	981	1031	1084	1139	1197	1256
16-#11	499.2			863	905	949	995	1043	1094	1147	1202	1259	1319
18-#11	561.6				967	1011	1057	1106	1156	1209	1264	1322	1381
20-#11	624.0					1074	1120	1168	1219	1272	1327	1384	1444
22-#11	686.4						1182	1230	1281	1334	1389	1446	1506
24-#11	748.8							1293	1343	1396	1451	1509	1568
26-#11	811.2								1406	1459	1514	1571	1631
28-#11	873.6							8%		1521	1576	1634	1693

Values below and to the left of the intermediate zigzag line require more vertical bars than can be accommodated in one ring (see page 249).

Outside diameter of spiral should be 3 in. less than the side of the column.

* To facilitate deliveries and eliminate errors, use only one grade of spiral on any one contract, generally available in the order intermediate grade hot rolled rod, cold drawn wire and hard grade hot rolled rod.

SPIRALLY REINFORCED SQUARE COLUMNS—Safe Axial Load in Kips

$$P = 1125\ A_g + 20{,}000\ A_s$$

28	29	30	31	32	33	34	35	36	37	38	39	40
Use Cold Drawn Wire												
⅝	⅝	⅝	⅝	⅝	⅝	⅝	Use Double Concentric Spirals					
2	2	2	2	2	2	2	$p'=$ .0196	.0193	.0190	.0188	.0185	.0183
882.0	946.1	1012.5	1081.1	1152.0	1225.1	1300.5	1378.1	1458.0	1540.1	1624.5	1711.1	1800.0
							CONCRETE $f'_c = 5{,}000$ psi HARD GRADE VERT. BARS					
1042												
1069	1133	1199										
1085	1149	1215	1284									
1131	1195	1262	1330	1401	1474	1550	1627					
1194	1258	1324	1393	1464	1537	1612	1690	1770	1852	1936	2023	1%
1256	1320	1386	1455	1526	1599	1674	1752	1832	1914	1998	2085	2174
1318	1382	1449	1517	1588	1661	1737	1814	1894	1976	2061	2147	2236
1381	1445	1511	1580	1651	1724	1799	1877	1957	2039	2123	2210	2299
1443	1507	1574	1642	1713	1786	1862	1939	2019	2101	2186	2272	2361
1506	1570	1636	1705	1776	1849	1924	2002	2082	2164	2248	2335	2424
1568	1632	1698	1767	1838	1911	1986	2064	2144	2226	2310	2397	2486
1630	1694	1761	1829	1900	1973	2049	2126	2206	2288	2373	2459	2548
1693	1757	1823	1892	1963	2036	2111	2189	2269	2351	2435	2522	2611
1755	1819	1886	1954	2025	2098	2174	2251	2331	2413	2498	2584	2673

SPIRALS AS PERCENTAGES OF CORE VOLUME (out to out of spirals)

See explanation on page 251.

Core Dia (in.)	⅝ in. Dia Pitch (in.)							½ in. Dia Pitch (in.)						
	2	2¼	2½	2¾	3	3¼	3½	2	2¼	2½	2¾	3	3¼	3½
11														
12	5.17							3.33						
13	4.77							3.08						
14	4.44	3.94						2.86	2.54					
15	4.13	3.68	3.31					2.67	2.37	2.13				
16	3.88	3.45	3.10					2.50	2.22	2.00				
17	3.65	3.24	2.92	2.65				2.35	2.09	1.88	1.71			
18	3.45	3.06	2.76	2.51	2.30			2.22	1.97	1.78	1.62	1.48		
19	3.26	2.90	2.61	2.37	2.18			2.11	1.87	1.68	1.53	1.40		
20	3.10	2.76	2.48	2.25	2.07	1.91		2.00	1.78	1.60	1.45	1.33	1.23	
21	2.95	2.63	2.36	2.15	1.97	1.82	1.69	1.90	1.69	1.52	1.38	1.27	1.17	1.09
22	2.82	2.51	2.26	2.05	1.88	1.74	1.61	1.82	1.62	1.45	1.32	1.21	1.12	1.04
23	2.70	2.40	2.16	1.96	1.80	1.66	1.54	1.74	1.55	1.39	1.26	1.16	1.07	0.99
24	2.59	2.30	2.07	1.88	1.72	1.59	1.48	1.67	1.48	1.33	1.21	1.11	1.03	0.95
25	2.48	2.21	1.98	1.80	1.65	1.53	1.42	1.60	1.42	1.28	1.16	1.07	0.98	0.91
26	2.39	2.12	1.91	1.73	1.59	1.47	1.36	1.54	1.37	1.23	1.12	1.03	0.95	0.88
27	2.30	2.04	1.84	1.67	1.53	1.41	1.31	1.48	1.32	1.19	1.08	0.99	0.91	0.85
28	2.21	1.97	1.77	1.61	1.48	1.36	1.27	1.43	1.27	1.14	1.04	0.95	0.88	0.82
29	2.14	1.90	1.71	1.55	1.43	1.32	1.22	1.38	1.23	1.10	1.00	0.92	0.85	0.79
30	2.07	1.84	1.65	1.50	1.38	1.27	1.18	1.33	1.19	1.07	0.97	0.89	0.82	0.76
31	2.00	1.78	1.60	1.45	1.33	1.23	1.14	1.29	1.15	1.03	0.94	0.86	0.79	0.73
32	1.94	1.72	1.55	1.41	1.29	1.19	1.11	1.25	1.11	1.00	0.90	0.83	0.77	0.71
33	1.88	1.67	1.50	1.37	1.25	1.15	1.07	1.21	1.08	0.97	0.88	0.80	0.74	0.69
34	1.82	1.62	1.46	1.33	1.22	1.12	1.04	1.18	1.04	0.94	0.85	0.78	0.72	0.67
35	1.77	1.58	1.42	1.29	1.18	1.09	1.01	1.14	1.02	0.91	0.83	0.76	0.70	0.65
36	1.72	1.53	1.38	1.25	1.15	1.06	0.98	1.11	0.98	0.89	0.80	0.74	0.68	0.63
37	1.67	1.49	1.34	1.22	1.12	1.03	0.96	1.08	0.96	0.86	0.78	0.72	0.66	0.61
38	1.63	1.45	1.30	1.19	1.09	1.00	0.93	1.05	0.94	0.84	0.76	0.70	0.65	0.60
39	1.59	1.41	1.27	1.16	1.06	0.98	0.91	1.02	0.91	0.82	0.74	0.68	0.63	0.58
40	1.55	1.38	1.24	1.13	1.03	0.95	0.88	1.00	0.89	0.80	0.72	0.66	0.61	0.57
41	1.51	1.34	1.21	1.10	1.01	0.93	0.86	0.98	0.87	0.78	0.71	0.65	0.60	0.55
42	1.48	1.31	1.18	1.07	0.98	0.91	0.84	0.95	0.85	0.76	0.69	0.63	0.58	0.54
43	1.44	1.28	1.15	1.05	0.96	0.88	0.82	0.93	0.83	0.74	0.67	0.62	0.57	0.53
44	1.41	1.25	1.13	1.02	0.94	0.86	0.80	0.91	0.81	0.72	0.66	0.60	0.56	0.52
45	1.38	1.22	1.10	1.00	0.92	0.85	0.79	0.89	0.79	0.71	0.64	0.59	0.54	0.50
46	1.35	1.20	1.08	0.98	0.90	0.83	0.77	0.87	0.77	0.69	0.63	0.58	0.53	0.49
47	1.32	1.17	1.06	0.96	0.88	0.81	0.75	0.85	0.76	0.68	0.62	0.56	0.52	0.48
48	1.29	1.15	1.03	0.94	0.86	0.79	0.74	0.83	0.74	0.66	0.60	0.55	0.51	0.47

SPIRALS AS PERCENTAGES OF CORE VOLUME (out to out of spirals)

See explanation on page 251.

⅜ in. Dia Pitch (in.)							¼ in. Dia Pitch (in.)					
1¾	2	2¼	2½	2¾	3	3¼	1¾	2	2¼	2½	2¾	3
2.29							1.04					
2.09	1.83						0.95	0.83				
1.93	1.69						0.88	0.77				
1.80	1.57	1.40					0.82	0.71	0.63			
1.68	1.47	1.30	1.17				0.76	0.67	0.59	0.53		
1.57	1.38	1.22	1.10				0.71	0.62	0.56	0.50		
1.48	1.29	1.15	1.03	0.94			0.67	0.59	0.52	0.47	0.43	
1.40	1.22	1.09	0.98	0.89	0.81		0.63	0.56	0.49	0.44	0.40	0.37
1.32	1.16	1.03	0.93	0.84	0.77		0.60	0.53	0.47	0.42	0.38	0.35
1.26	1.10	0.98	0.88	0.80	0.73	0.68	0.57	0.50	0.44	0.40	0.36	0.33
1.20	1.05	0.93	0.84	0.76	0.70	0.64	0.54	0.48	0.42	0.38	0.35	0.32
1.14	1.00	0.89	0.80	0.73	0.67	0.62	0.52	0.45	0.40	0.36	0.33	0.30
1.09	0.96	0.85	0.76	0.70	0.64	0.59	0.50	0.43	0.39	0.35	0.32	0.29
1.05	0.92	0.81	0.73	0.67	0.61	0.56	0.48	0.42	0.37	0.33	0.30	0.28
1.01	0.88	0.78	0.70	0.64	0.59	0.54	0.46	0.40	0.36	0.32	0.29	0.27
0.97	0.85	0.75	0.68	0.62	0.56	0.52	0.44	0.38	0.34	0.31	0.28	0.26
0.93	0.82	0.72	0.65	0.59	0.54	0.50	0.42	0.37	0.33	0.30	0.27	0.25
0.90	0.79	0.70	0.63	0.57	0.52	0.48	0.41	0.36	0.32	0.29	0.26	
0.87	0.76	0.67	0.61	0.55	0.51	0.47	0.39	0.34	0.31	0.28	0.25	
0.84	0.73	0.65	0.58	0.53	0.49	0.45	0.38	0.33	0.29	0.26		
0.81	0.71	0.63	0.56	0.51	0.47	0.43	0.37	0.32	0.28	0.25		
0.78	0.68	0.61	0.55	0.50	0.46	0.42	0.36	0.31	0.27	0.25		
0.76	0.66	0.59	0.53	0.48	0.44	0.41	0.34	0.30	0.27			
0.74	0.64	0.57	0.51	0.47	0.43	0.39	0.33	0.29	0.26			
0.71	0.62	0.56	0.50	0.46	0.42	0.38	0.32	0.28	0.25			
0.69	0.61	0.54	0.49	0.44	0.40	0.37	0.31	0.27				
0.68	0.59	0.52	0.47	0.43	0.39	0.36	0.30	0.27				
0.66	0.57	0.51	0.46	0.42	0.38	0.35	0.30	0.26				
0.64	0.56	0.50	0.45	0.41	0.37	0.34	0.29	0.25				
0.62	0.55	0.49	0.44	0.40	0.36	0.34	0.28	0.25				
0.61	0.53	0.47	0.43	0.39	0.35	0.33	0.27					
0.59	0.52	0.46	0.42	0.38	0.35	0.32	0.27					
0.58	0.51	0.45	0.41	0.37	0.34	0.31	0.26					
0.57	0.50	0.44	0.40	0.36	0.33	0.31	0.26					
0.55	0.48	0.43	0.39	0.35	0.32	0.30	0.25					
0.54	0.47	0.42	0.38	0.34	0.32	0.29						
0.53	0.46	0.41	0.37	0.34	0.31	0.28						
0.52	0.45	0.40	0.36	0.33	0.30	0.28						

WEIGHTS OF SPIRALS PER VERTICAL FOOT *

See explanation on page 251.

Core Dia (in.)	⅝ in. Dia Pitch (in.)							½ in. Dia Pitch (in.)						
	2	2¼	2½	2¾	3	3¼	3½	2	2¼	2½	2¾	3	3¼	3½
11														
12	19.7							12.6						
13	21.3							13.6						
14	22.9	20.4						14.7	13.0					
15	24.6	21.8	19.7					15.7	14.0	12.6				
16	26.2	23.3	21.0					16.8	14.9	13.4				
17	27.9	24.8	22.3	20.3				17.8	15.9	14.3	13.0			
18	29.5	26.2	23.6	21.4	19.7			18.9	16.8	15.1	13.7	12.6		
19	31.1	27.7	24.9	22.6	20.8			20.0	17.7	16.0	14.5	13.3		
20	32.8	29.1	26.2	23.8	21.8	20.2		21.0	18.6	16.8	15.3	14.0	12.9	
21	34.4	30.6	27.5	25.0	22.9	21.2	19.7	22.0	19.6	17.6	16.0	14.7	13.6	12.6
22	36.0	32.0	28.8	26.2	24.0	22.2	20.6	23.1	20.5	18.5	16.8	15.4	14.2	13.2
23	37.7	33.5	30.1	27.4	25.1	23.2	21.5	24.1	21.4	19.3	17.6	16.1	14.9	13.8
24	39.3	35.0	31.5	28.6	26.2	24.2	22.5	25.2	22.4	20.2	18.3	16.8	15.5	14.4
25	41.0	36.4	32.8	29.8	27.3	25.2	23.4	26.2	23.3	21.0	19.1	17.5	16.1	15.0
26	42.6	37.9	34.1	31.0	28.4	26.2	24.3	27.3	24.3	21.8	19.9	18.2	16.8	15.6
27	44.2	39.3	35.4	32.2	29.5	27.2	25.3	28.3	25.2	22.7	20.6	18.9	17.4	16.2
28	45.9	40.8	36.7	33.4	30.6	28.2	26.2	29.4	26.1	23.5	21.4	19.6	18.1	16.8
29	47.5	42.2	38.0	34.6	31.7	29.2	27.2	30.4	27.1	24.4	22.2	20.3	18.7	17.4
30	49.2	43.7	39.3	35.7	32.8	30.2	28.1	31.5	28.1	25.2	22.9	21.0	19.4	18.0
31	50.8	45.3	40.5	36.9	33.9	31.2	29.0	32.5	29.0	26.1	23.7	21.7	20.1	18.6
32	52.5	46.7	41.7	38.1	35.0	32.2	29.9	33.6	29.9	26.9	24.4	22.4	20.7	19.2
33	54.1	48.1	43.1	39.3	36.1	33.2	30.9	34.6	30.9	27.7	25.2	23.1	21.4	19.8
34	55.7	49.6	44.4	40.5	37.2	34.2	31.9	35.7	31.9	28.6	26.0	23.8	22.0	20.4
35	57.4	51.0	45.7	41.7	38.3	35.2	32.8	36.7	32.8	29.4	26.7	24.5	22.7	21.0
36	59.1	52.5	47.0	42.8	39.4	36.2	33.7	37.8	33.7	30.3	27.5	25.2	23.3	21.6
37	60.7	53.9	48.3	44.1	40.5	37.3	34.7	38.8	34.7	31.1	28.3	25.9	24.0	22.2
38	62.3	55.4	49.6	45.2	41.6	38.3	35.6	39.9	35.6	31.9	29.0	26.6	24.6	22.8
39	63.9	56.8	50.9	46.4	42.7	39.3	36.5	40.9	36.5	32.8	29.8	27.3	25.2	23.4
40	65.6	58.3	52.2	47.6	43.8	40.3	37.5	42.0	37.5	33.6	30.6	28.0	25.9	24.0
41	67.3	59.7	53.5	48.8	44.8	41.3	38.4	43.0	38.4	34.4	31.3	28.7	26.6	24.6
42	68.9	61.2	54.8	50.0	45.9	42.3	39.3	44.1	39.3	35.3	32.1	29.4	27.2	25.2
43	70.5	62.7	56.1	51.1	47.0	43.3	40.3	45.1	40.3	36.1	32.8	30.1	27.8	25.8
44	72.1	64.1	57.5	52.4	48.1	44.3	41.2	46.2	41.2	37.0	33.6	30.8	28.5	26.4
45	73.8	65.5	58.7	53.6	49.2	45.3	42.2	47.3	42.2	37.8	34.4	31.5	29.1	27.0
46	75.5	67.0	60.1	54.7	50.3	46.3	43.1	48.3	43.1	38.6	35.1	32.2	29.8	27.6
47	77.1	68.5	61.4	55.9	51.4	47.3	44.1	49.4	44.1	39.5	35.9	32.9	30.4	28.2
48	78.7	70.0	62.7	57.1	52.5	48.3	45.0	50.4	45.0	40.4	36.7	33.6	31.1	28.8

* The weights given include wire for regular loops only. Weight must be added for 1½ turns top and bottom required for embedment, equivalent to one-half the tabular weight for 2-in. pitch. Weight of spacers must also be added. A ⅞-in. channel spacer weighs ¾ lb per lin. ft. Two spacers are required for spirals 20 in. or less in diameter, three for spirals 20 to 30 in., and four for spirals over 30 in. in diameter.

WEIGHTS OF SPIRALS PER VERTICAL FOOT *

See explanation on page 251.

⅜ in. Dia Pitch (in.)							¼ in. Dia Pitch (in.)					
1¾	2	2¼	2½	2¾	3	3¼	1¾	2	2¼	2½	2¾	3
7.43							3.30					
8.10	7.09						3.60	3.15				
8.78	7.68						3.90	3.41				
9.45	8.27	7.35					4.20	3.67	3.26			
10.1	8.85	7.87	7.08				4.50	3.94	3.50	3.15		
10.8	9.44	8.39	7.55				4.80	4.20	3.73	3.36		
11.5	10.0	8.92	8.02	7.30			5.10	4.46	3.96	3.56	3.24	
12.1	10.6	9.44	8.50	7.73	7.08		5.40	4.72	4.20	3.78	3.43	3.15
12.8	11.2	9.97	8.97	8.15	7.48		5.70	4.98	4.43	3.98	3.62	3.32
13.5	11.8	10.5	9.44	8.59	7.87	7.28	6.00	5.25	4.66	4.20	3.82	3.50
14.2	12.4	11.0	9.92	9.02	8.26	7.64	6.30	5.51	4.90	4.41	4.01	3.67
14.8	13.0	11.5	10.4	9.45	8.66	8.00	6.60	5.77	5.13	4.62	4.20	3.85
15.5	13.6	12.1	10.9	9.88	9.05	8.36	6.90	6.03	5.36	4.83	4.39	4.02
16.2	14.2	12.6	11.3	10.3	9.45	8.72	7.20	6.30	5.60	5.04	4.58	4.20
16.9	14.8	13.1	11.8	10.7	9.84	9.09	7.50	6.56	5.83	5.25	4.77	4.37
17.5	15.4	13.6	12.3	11.2	10.2	9.45	7.80	6.82	6.06	5.46	4.96	4.55
18.2	15.9	14.2	12.7	11.6	10.6	9.81	8.10	7.08	6.30	5.67	5.15	4.72
18.9	16.5	14.7	13.2	12.0	11.0	10.2	8.40	7.34	6.53	5.88	5.34	
19.6	17.1	15.2	13.7	12.5	11.4	10.5	8.70	7.60	6.77	6.09	5.53	
20.2	17.7	15.7	14.2	12.9	11.8	10.9	8.99	7.87	7.00	6.30		
20.9	18.3	16.2	14.7	13.4	12.2	11.3	9.14	8.13	7.23	6.51		
21.6	18.9	16.8	15.2	13.8	12.6	11.7	9.57	8.39	7.46	6.72		
22.3	19.5	17.3	15.6	14.2	13.0	12.0	9.87	8.66	7.70			
22.9	20.1	17.8	16.1	14.6	13.4	12.4	10.2	8.92	7.93			
23.6	20.7	18.3	16.6	15.1	13.8	12.8	10.5	9.18	8.17			
24.3	21.3	18.9	17.1	15.5	14.2	13.1	10.8	9.44				
24.9	21.9	19.4	17.5	15.9	14.6	13.5	11.1	9.71				
25.6	22.4	19.9	18.0	16.4	15.0	13.8	11.4	9.97				
26.3	23.0	20.4	18.5	16.8	15.4	14.2	11.7	10.25				
27.0	23.6	21.0	19.0	17.2	15.8	14.6	12.0	10.50				
27.6	24.2	21.5	19.4	17.7	16.2	14.9	12.3					
28.3	24.8	22.0	19.9	18.1	16.6	15.3	12.6					
29.0	25.4	22.5	20.4	18.5	16.9	15.7	12.9					
29.6	26.0	23.0	20.8	18.9	17.3	16.0	13.2					
30.3	26.6	23.6	21.3	19.4	17.7	16.4	13.5					
31.0	27.2	24.1	21.8	19.8	18.1	16.8						
31.7	27.8	24.6	22.3	20.2	18.5	17.1						
32.4	28.3	25.1	22.7	20.7	18.9	17.5						

* The weights given include wire for regular loops only. Weight must be added for 1½ turns top and bottom required for embedment, equivalent to one-half the tabular weight for 2-in. pitch. Weight of spacers must also be added. A ⅞-in. channel spacer weighs ¾ lb per lin. ft. Two spacers are required for spirals 20 in. or less in diameter, three for spirals 20 to 30 in., and four for spirals over 30 in. in diameter.

SOIL LOAD TEST

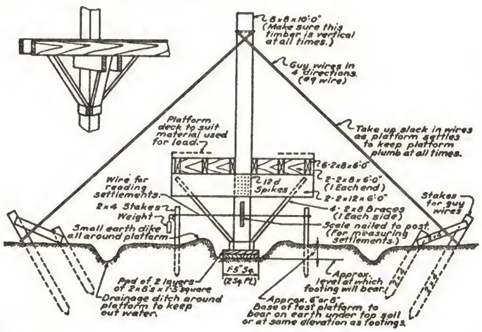
Information on subsoil conditions is best obtained by:—

(1) Observation of nearby structures, and a determination of their bearing intensity and amount of settlement.

(2) Digging an open pit to a depth below the desired bearing level and judging the soil strata on the basis of familiarity with local soils.

(3) Exploratory borings, driving a casing by means of a drop weight (frequently 140 pounds dropping 30 in.), recording the number of blows to drive the casing a known distance, and comparing with data from other borings, to approximate the safe bearing capacity; also the obtaining of undisturbed samples from within the casing and, for clay, testing triaxial shear and free column compression as an added method of estimating safe bearing value.

(4) Applying physical load test to a square (sometimes circular) bearing block directly on the stratum where footings will rest, applied in the following manner:—



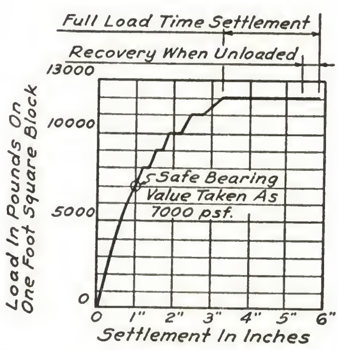
Method A (proving soil capacity):—

Assume a design load.
Build platform like sketch.
Load platform with assumed design load.
Leave until periodic measurements show no further settlement for a period of 24 hours.
Add four increments (each equal to 25 per cent of the design load) at 4-hour intervals.
Leave total (double design) load until no measurable settlement occurs during a period of 24 hours.
Readings are to be taken to $\frac{1}{32}$ in. every hour for the first 6 hours and every 12 hours thereafter.
When design load causes settlement of less than $\frac{3}{8}$ in. and twice the design load of less than 1 in., the design load shall be allowed (but medium clay shall not exceed 4 tons psf, or soft clay 1 ton psf).

Method B (investigating soil capacity):—

When design load is not assumed in advance, place load in increments of, say, 2000 pounds or appropriate to the total load.
Add no further increment until readings at half-hour intervals show no further settlement at end of one hour.
Add second increment and check every half hour until there is no further settlement for a period of one hour.
Continue increments and measurements until total applied load is at least double the desired design load or until excessive settlement begins to occur.
If possible, leave load 48 hours, taking readings every 8 hours.
Remove the load, measure the recovery, plot a load-time-settlement curve to determine safe bearing value.

The accompanying graph shows such a curve, with slight time settlements at loads above 7000 psf, and a decided time settlement at 12,000 psf, and only a moderate recovery. In this case, 7000 psf was selected as a fairly high safe bearing value from such a curve.



ECCENTRICALLY LOADED CONCRETE COLUMNS

These tables are adapted from those of I. E. Morris * and give a very rapid solution of that difficult problem of a reinforced concrete member undergoing combined bending and direct compression. This condition arises in arch ribs, where flexural members frame monolithically into their supporting columns, where loads are applied to columns through brackets as in the case of crane girders, and in many similar situations. For the determination of column moments in rigid frames, see page 75.

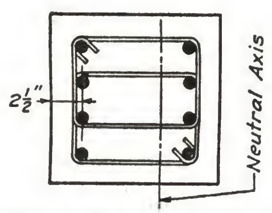
Three sets of tables are given:—(1) for square tied columns, (2) for spirally reinforced square columns, and (3) for spirally reinforced round columns. Two grades of concrete are tabulated; $f'_c = 3000$ psi and 3750 psi. All practicable combinations of vertical steel are included.

Loads are expressed in kips (1000-lb units). Eccentricities of the applied load, N, are given in inches from 0 to 12 or 14. The vertical steel is given in the column headed "bars." The tables can obviously be used equally well for concentric loads by taking the values in the column "$e = 0$." If the applied moment is given instead of the eccentricity of the load, simply divide M in lb-in. by N in pounds to get e in inches.

SECTION I—SQUARE TIED COLUMNS

In the section dealing with square tied columns, only one fiber stress is used in the vertical bars, viz., $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi (applicable to hard grade bars), but a reduction factor is given at the bottom of each column in the table when it is necessary to use a stress of $f_s = 0.8 \times 16{,}000 = 12{,}800$ psi † (applicable to intermediate grade bars). This reduction factor involves an error of not over about 3 per cent. For those desiring greater precision, tables giving $B = CD$ values for a stress of $f_s = 12{,}800$ psi are included at the end of this section and $B/t = CD/t$ values are tabulated for each column listed. See ACI Code 1104, 1107-1109 for the methods and formulas used here.

One-half of the bars given in the table are to be placed in each of the two faces of the column which are perpendicular to the plane of bending (see figure below).



One-half of total vertical steel in each of two col. faces perpendicular to the plane of bending.

* "Allowable Loads on Eccentrically Loaded Concrete Columns" by I. E. Morris, Atlanta, Ga., 1947.

† In a symmetrical, uncracked section, the compressive stress will exceed the tensile stress and the allowable f_s will be that for columns and not for flexure.

ECCENTRICALLY LOADED SQUARE TIED CONCRETE COLUMNS

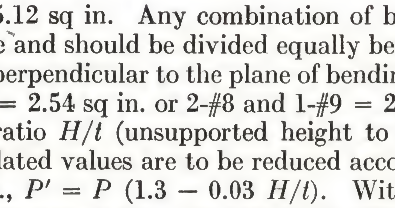
The values given for square tied columns may be easily extended to include rectangular tied columns as follows:—Assume the load to be supported is 100 kips with an eccentricity of 4 in. on a column whose sides parallel to the plane of bending are limited to 12 in. Table on page 281 shows that a 12 × 12 column reinforced with 4-#8 bars will carry 62 kips with a 4 in. eccentricity. Accordingly, the sides of the column perpendicular to the plane of bending should be $\left(\frac{100}{62} = 1.62\right) \times 12$ in., or 19.44 in., say, 12 × 20. For the steel area, take $1.62 \times 4 \times 0.79 = 5.12$ sq in. Any combination of bars that will give this area will be adequate and should be divided equally between the two faces of the column that are perpendicular to the plane of bending. Possible selections include either 2-#10 = 2.54 sq in. or 2-#8 and 1-#9 = 2.58 sq in. in each face.

Whenever the ratio H/t (unsupported height to least side of column) exceeds 10, the tabulated values are to be reduced according to the ACI long column formula, viz., $P' = P\ (1.3 - 0.03\ H/t)$. With eccentrically loaded columns, H/t is not to exceed 20. For tabulation of values, see page 233.

The spacing of ties in these tied columns can be taken from the corresponding tables for axially loaded square tied columns in the preceding section, pages 236 to 247, inclusive. For the few cases where bending predominates, shear can readily be checked.

While the scope of these tables is quite complete, some illustrative examples are shown for those who wish to design beyond the range of the tables or to see how they were prepared.

Example I—For the table on page 283, verify the value $N = 206$ kips with an eccentricity of 3 in. for a 20 in. square tied column of 3000 psi concrete reinforced with 6-#8 vertical bars, using $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi, $f_c = 0.8 \times 0.225\ f'_c = 540$ psi, and $n = 10$.



To verify the tabular value of N, determine whether the sum of the ratio of nominal direct stress to the nominal allowed compressive stress and the ratio of computed bending stress to allowed bending stress $\lessgtr 1$. Since the point of application is well within the middle third of the 20 x 20 section, the load will act within the kern of the transformed section,* producing compression over the entire area. Solve by the elastic theory using the transformed section:—

Section	*Area of Transformed Section*	*Capacity (Concentric Axial Loading)*	*Moment of Inertia*
Concrete	$20 \times 20 = 400.0$	$400 @ 540 = 216{,}000$	$400 \times \frac{20^2}{12} = 13{,}330$
Bars	$2 \times 21.3 = 42.6$	$6 \times 0.79 @ 16{,}000 = 75{,}800$	$42.6 \times 7.5^2 = 2{,}400$
	442.6 sq in.	291,800 lb	15,730 in.⁴

$$\text{Unit direct stress} = \frac{N}{A} = \frac{206{,}000}{442.6} = 466 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = \frac{206{,}000 \times 3 \times 10}{15{,}730} = 393 \text{ psi}$$

859 psi Max Comp
73 psi Min Comp

* See page 93.

ECCENTRICALLY LOADED SQUARE TIED CONCRETE COLUMNS

Neutral axis is $\dfrac{73 \times 20}{786} = 1.86$ in. outside face of column.

$f'_c = 10\,\dfrac{17.5 + 1.86}{21.86} \times 859 = 7610$ psi Comp.

Average allowable compressive stress with axial load, $\dfrac{P}{A} = \dfrac{291{,}800}{442.6} = 659$ psi.

By the method of ACI 1109a, $\dfrac{f_a}{F_a} + \dfrac{f_b}{F_b} \leqq 1$, and cancelling A_g in the denominators of both f_a and F_a, we can write $\dfrac{N}{P} + \dfrac{f_b}{F_b} \leqq 1$, or $\dfrac{206{,}000}{291{,}800} + \dfrac{393}{1350} \leqq 1$, so $0.707 + 0.293 = 1$.

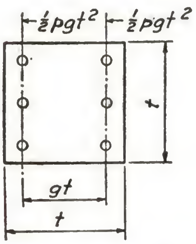
A somewhat lengthier method was used in the 1951 ACI code to get the same result. Since these tables are based upon the use of hard grade bars and the conversion to intermediate grade is facilitated by the tabulated *CD* (now called B) factors of the previous code, their use is explained. The allowable eccentric load may be determined from $N = \dfrac{P}{\left(1 + \dfrac{CDe}{t}\right)}$ [which is equivalent to $N = \dfrac{P}{\left(1 + \dfrac{Be}{t}\right)}$ (ACI 1109c)]

where P is the capacity for concentric axial load, e is the eccentricity, t is the column width perpendicular to the axis of bending, C is the ratio of allowable compressive stress in axially loaded column to allowable bending stress, and the D term is obtained as follows:—

$$f_c = \frac{N}{A} + \frac{Nec}{I} = \frac{N}{A}\left(1 + \frac{et}{2R^2}\right) \text{ where } A = A_g + (n-1)\,p_g A_g.$$

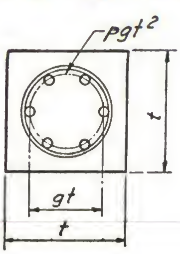
The term $\dfrac{et}{2R^2} = \dfrac{et^2}{2tR^2} = \dfrac{De}{t}$. For a square unreinforced column, $D = 6$.

CASE I—Square Tied Column:—



$$D = \frac{t^2}{2R^2} = \frac{t^2A}{2I} = \frac{t^2}{2}\,\frac{[t^2 + (n-1)p_g t^2]}{\left[\dfrac{t^4}{12} + (n-1)p_g t^2\dfrac{(gt)^2}{4}\right]} = \frac{1 + (n-1)p_g}{\dfrac{1}{6} + (n-1)\dfrac{p_g g^2}{2}}$$

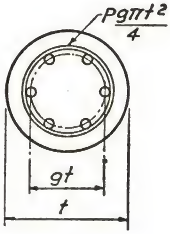
CASE II—Square Spiralled Column:—



$$D = \frac{t^2}{2}\,\frac{[t^2 + (n-1)p_g t^2]}{\left[\dfrac{t^4}{12} + \dfrac{(n-1)p_g t^2\left(\dfrac{gt}{2}\right)^2}{2}\right]} = \frac{1 + (n-1)p_g}{\dfrac{1}{6} + (n-1)\dfrac{p_g g^2}{4}}$$

ECCENTRICALLY LOADED SQUARE TIED CONCRETE COLUMNS

CASE III—Round Spiralled Column:—



$$D = \frac{t^2}{2}\frac{\left[\frac{\pi t^2}{4} + (n-1)\frac{p_g \pi t^2}{4}\right]}{\left[\frac{\pi t^4}{64} + \frac{(n-1)p_g \pi t^2 \left(\frac{gt}{2}\right)^2}{8}\right]} = \frac{1 + (n-1)p_g}{\frac{1}{8} + (n-1)\frac{p_g g^2}{4}}$$

Example I—*Second Solution*

The verification of Ex. I can also be made by using the formula just explained:

$N = \frac{P}{\left(1 + \frac{CDe}{t}\right)} = \frac{P}{\left(1 + \frac{Be}{t}\right)}$. Using the formulas derived for Case I on page 277,

compute g (the figure on page 277) as $\frac{15}{20} = 0.75$, and p_g as $\frac{6 \times 0.79}{20 \times 20} = 0.01185$, then

$$D = \frac{1 + (n-1)p_g}{\frac{1}{6} + 0.5\,(n-1)p_g g^2} = \frac{1 + 0.1067}{0.1667 + 0.5 \times 0.1067 \times 0.75^2} = 5.63\text{, and}$$

$$C = \frac{P/A}{0.45f'_c} = \frac{659}{1350} = 0.489$$

$\frac{B}{t} = \frac{CD}{t} = 0.489 \times \frac{5.63}{20} = 0.138$. This can also be obtained directly from page 283 opposite 6-#8 bars in the column headed CD/t.

$$N = \frac{P}{\left(1 + \frac{CDe}{t}\right)} = \frac{P}{\left(1 + \frac{Be}{t}\right)} = \frac{291{,}800}{(1 + 0.138 \times 3)} = 206 \text{ kips}$$

Example II—Use the same data as in Ex. I except reduce the steel stress in the vertical bars to $f_s = 0.8 \times 16{,}000 = 12{,}800$ psi, and check the approximate value $N = 0.940 \times 206 = 193$ kips.

Concrete	20 × 20 @ 540	= 216,000 lb
Bars	6 × 0.79 @ 12,800 =	60,600 lb
		276,600 lb

$$\frac{P}{A_g} = \frac{276{,}600}{400} = 692 \text{ psi}$$

Then $\frac{N/A_g}{P/A_g} + \frac{Nec/I}{F_b} = \frac{193{,}000/400}{692} + \frac{193{,}000 \times 3 \times 10/15{,}730}{1350} =$

$0.700 + 0.273 = 0.973 < 1.00$

so the value obtained in this way is 2¾ per cent below the allowable.

Example IIa—Solve Ex. II using $B = CD$ values on page 358.

As before, $p = 0.01185$ and $g = 0.75$, so from the table for $f'_c = 3000$, $B = CD = 2.58$,

and $N = \frac{P}{\left(1 + \frac{CDe}{t}\right)} = \frac{P}{\left(1 + \frac{Be}{t}\right)} = \frac{276{,}600}{\left(1 + \frac{2.58 \times 3}{20}\right)} = 199{,}000$

To check: $\frac{199{,}000/400}{692} + \frac{199{,}000 \times 3 \times 10/15{,}730}{1350} = 0.720 + 0.281 = 1.001$,

so using the $B = CD$ value is more precise than using the approximate factors.

ECCENTRICALLY LOADED SQUARE TIED CONCRETE COLUMNS

Example III—If the column in Ex. I had been 26′-8″ high, what effect would this have had on the value $N = 206$ kips allowed when $H/t \gtrless 10$?

$$\frac{H}{t} = \frac{26.67}{1.67} = 16 \qquad \left(1.3 - 0.03\frac{H}{t}\right) = 0.82\ *$$

Then the safe allowable eccentric load, $N = 0.82 \times 206 = 168.9$ kips.

Example IV—For the same data as in Ex. I, increase the eccentricity of the load to 7 in. so that the resultant will definitely be outside of the kern of the section, yet definitely within the two-thirds of t limit placed by ACI 1109(a) and (d), and check the value $N = 148$ kips in the table on page 283.

ACI 1109(a) states that when the ratio of eccentricity e/t does not exceed ⅔, the combined fiber stress in compression may be computed on the basis of recognized theory applying to uncracked sections. This means that the formula for combined bending and direct stress may be applied for a load outside of the kern of the section, provided that the maximum eccentricity does not exceed two-thirds the width of the member, t, and that tension may be considered as developing in the concrete on the side away from the eccentrically applied load. Then:—

$$\text{Unit direct stress} = \frac{N}{A} = \frac{148{,}000}{442.6} = 334 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = \frac{148{,}000 \times 7 \times 10}{15{,}730} = \underline{658} \text{ psi}$$

$$\text{Actual stresses} \begin{cases} 992 \text{ psi Max Comp} \\ 324 \text{ psi Max Tens} \end{cases}$$

Since either nominal direct stress or actual direct stress will produce the same results if used consistently in both numerator and denominator and since we already have the actual direct stresses, use them in the formula $\frac{f_a}{F_a} + \frac{f_b}{F_b} \gtrless 1$:—

$$\frac{334}{659} + \frac{658}{1350} = 0.507 + 0.488 = 0.995 < 1.00$$

Another method, as in the second solution of Ex. I, using the values developed there is:—

$$N = \frac{P}{\left(1 + \frac{Be}{t}\right)} = \frac{291{,}800}{(1 + 0.138 \times 7)} = 148{,}000 \text{ lb}$$

Example IVa—To see the effect of applying the theory of the cracked section (no tension in the concrete), work Ex. IV on the basis of a cracked section, although according to ACI 1109(d) this method is not to be applied for $e < 2/3t$.

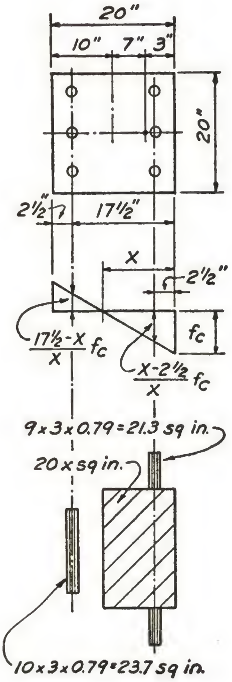
From the figure on page 280, ΣM about the point of application of the load must equal zero, so:—

$$-\begin{bmatrix}\text{Conc. Comp.}\\ \frac{1}{2}f_c 20x\end{bmatrix}\begin{bmatrix}\text{Arm}\\ \frac{x}{3} - 3\end{bmatrix} + \begin{bmatrix}\text{Steel Comp.}\\ f_c\frac{x-2.5}{x}21.3\end{bmatrix}\begin{bmatrix}\text{Arm}\\ \frac{1}{2}\end{bmatrix} + \begin{bmatrix}\text{Tension}\\ f_c\frac{17.5-x}{x}23.7\end{bmatrix}\begin{bmatrix}\text{Arm}\\ 14.5\end{bmatrix} = 0$$

$$x^3 - 9x^2 + 99.7x - 1797 = 0 \qquad x = 12.5 \text{ in.}$$

* This value can also be taken from the table on page 233.

ECCENTRICALLY LOADED SQUARE TIED CONCRETE COLUMNS



Also, ΣV must be zero:—

$$\frac{1}{2}f_c\,20x + f_c\frac{x-2.5}{x}\,21.3 - f_c\frac{17.5-x}{x}\,23.7 - 148{,}000 = 0$$

$$125f_c + 17.04f_c - 9.48f_c = 148{,}000$$

$$f_c = \frac{148{,}000}{132.6} = 1115 \text{ psi Max Comp}$$

$$f_s = 1115 \times 10 \times \frac{10}{12.5} = 8940 \text{ psi Comp}$$

$$f_s = 1115 \times 10 \times \frac{5}{12.5} = 4470 \text{ psi Tens}$$

The unit direct stress being 334 psi (Ex. IV), the unit bending stress must be $1115 - 334 = 781$ psi, and $\frac{334}{659} + \frac{781}{1350} = 0.507 + 0.578 = 1.085 > 1.000$

From these figures, the maximum actual compressive stress, neglecting tension in the concrete on the opposite side of the neutral axis, is somewhat greater than the maximum allowable. The maximum compressive stress is not too greatly affected whether or not we include tension in the concrete, but on the opposite side of the column the minimum compression or maximum tension will differ very greatly.

Ex. IV (uncracked section) indicates 324 psi tension in the extreme fiber of the concrete, which is in the range of the rather undependable ultimate tensile strength. Ex. IVa (cracked section), transfers this to a 4470 psi tension in the reinforcement. The corresponding change in the arm of the internal couple is what changes the compressive stress somewhat.

Also the compression steel will tend to carry twice its elastic value, shifting the neutral axis and relieving the situation somewhat.

Values to the left of the vertical line $\left(e \lessgtr \frac{2t}{3}\right)$ in the table are based upon uncracked sections; those to the right of the vertical line upon cracked sections. In those few cases to the right of the vertical line where an uncracked section produces lesser capacity, the lesser value is here used.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—12″ x 12″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#6	.0122	.240	106	85	71	61	54	48	43	39	36	26	23	21	18	16	15
4-#7	.0167	.265	116	92	77	65	57	51	46	41	38	30	27	24	22	20	19
4-#8	.0220	.272	128	101	82	70	62	54	48	43	40	34	31	28	25	23	22
4-#9	.0278	.288	142	109	89	75	65	57	51	46	42	39	35	31	29	26	24
4-#10	.0355	.296	159	121	98	83	71	63	56	50	46	43	39	36	33	30	28
For $f_s = 0.8 \times 16{,}000 = 12{,}800$ psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	†	†	†	†	†	†

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—14″ x 14″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#7	.0122	.207	144	120	102	89	79	71	65	59	55	51	41	37	34	30	27
4-#8	.0162	.214	157	129	110	95	84	76	69	63	58	53	47	42	39	36	33
4-#9	.0204	.223	170	139	118	102	90	80	73	66	61	56	52	48	43	40	37
4-#10	.0258	.234	187	152	127	110	97	86	78	71	65	60	56	52	49	45	42
4-#11	.0322	.242	206	165	138	118	104	92	83	76	69	64	60	56	53	50	47
6-#6	.0135	.212	148	123	105	91	81	73	66	60	56	52	43	39	35	32	29
6-#7	.0183	.217	164	136	114	99	87	78	71	64	59	55	50	45	41	38	35
6-#8	.0242	.230	182	148	125	108	95	85	77	70	65	60	55	52	47	44	41
6-#9	.0306	.240	202	162	136	117	103	91	82	75	69	63	59	55	52	49	46
6-#10	.0390	.254	228	182	150	129	113	100	90	81	75	69	64	60	57	53	50
For $f_s = 0.8 \times 16{,}000 = 12{,}800$ psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	†	†	†	†	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† Below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—16″ x 16″

Bars *	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#8	.0124	.176	189	161	140	124	111	100	92	85	78	73	68	60	55	50	45
4-#9	.0156	.179	202	171	149	132	118	106	97	90	83	77	72	67	61	56	52
4-#10	.0198	.189	219	184	159	140	125	112	103	94	87	81	76	71	67	63	59
4-#11	.0245	.198	238	199	170	149	133	120	109	100	92	85	80	75	70	67	63
6-#6	.0103	.173	180	153	134	118	106	96	88	81	75	70	66	55	50	41	38
6-#7	.0141	.178	196	166	145	128	114	104	95	87	81	75	70	64	58	54	50
6-#8	.0185	.186	214	181	156	137	123	111	101	93	86	80	75	70	66	61	57
6-#9	.0235	.196	234	196	168	147	131	118	108	99	91	85	79	74	70	66	62
6-#10	.0297	.201	260	216	186	162	144	130	118	108	100	93	86	81	76	72	68
6-#11	.0365	.209	288	238	203	177	157	141	128	117	108	100	93	87	82	77	73
8-#6	.0137	.178	194	165	143	126	113	103	94	86	80	75	70	63	58	53	49
8-#7	.0188	.187	215	181	156	138	123	111	101	93	86	80	75	70	66	62	57
8-#8	.0247	.198	239	200	171	150	133	120	109	100	92	86	80	75	71	67	63
8-#9	.0312	.203	266	221	189	165	147	132	120	110	101	94	88	82	77	73	69
8-#10	.0397	.211	301	248	212	184	163	146	133	122	112	104	97	90	85	80	76
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	†	†	†	†

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—18″ x 18″

Bars *	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#9	.0123	.156	239	207	182	163	147	134	123	114	106	99	93	88	83	76	70
4-#10	.0157	.160	256	220	194	173	156	142	130	121	112	105	98	93	88	83	79
4-#11	.0192	.167	275	236	206	183	165	150	137	127	118	110	103	97	92	87	82
6-#7	.0111	.154	233	202	178	159	144	132	121	112	104	98	92	86	82	72	66
6-#8	.0146	.159	251	217	192	170	153	140	128	119	110	103	97	91	86	82	76
6-#9	.0185	.165	271	232	204	181	163	148	136	126	117	109	102	96	91	86	82
6-#10	.0235	.175	297	253	220	195	175	158	145	134	124	115	108	102	96	91	86
6-#11	.0289	.178	325	276	240	212	190	172	157	145	134	125	117	110	104	98	93
8-#6	.0109	.153	231	200	177	158	143	131	120	112	104	97	91	86	81	71	65
8-#7	.0148	.159	252	217	191	170	154	140	129	119	111	104	97	92	87	82	76
8-#8	.0195	.167	276	236	207	184	165	150	138	127	118	110	103	97	92	87	83
8-#9	.0247	.175	303	258	224	199	178	162	148	136	126	118	110	104	98	92	88
8-#10	.0313	.180	338	286	248	219	196	178	162	149	138	129	121	113	107	101	96
8-#11	.0385	.186	375	316	273	241	215	194	177	163	151	140	131	123	116	110	104
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	†	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—20″ x 20″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#9	.0100	.136	280	246	220	199	181	167	154	143	134	126	119	112	106	101	91
4-#10	.0127	.138	297	261	233	210	191	176	162	151	141	132	125	118	112	106	101
4-#11	.0156	.141	316	277	246	222	202	185	171	159	148	139	131	124	117	112	106
6-#8	.0118	.138	292	256	229	206	188	173	160	148	139	130	123	116	110	104	98
6-#9	.0150	.140	312	274	244	220	200	183	170	158	147	138	130	123	116	111	105
6-#10	.0190	.146	338	295	262	235	213	195	180	167	156	146	137	130	123	117	111
6-#11	.0234	.151	366	318	281	252	228	208	192	178	166	155	146	137	130	123	118
8-#7	.0120	.138	293	257	230	207	189	173	160	149	139	131	123	116	110	105	99
8-#8	.0158	.141	317	278	247	223	203	186	172	160	149	140	132	124	118	112	107
8-#9	.0200	.147	344	300	266	239	216	198	183	169	158	148	139	132	124	118	112
8-#10	.0254	.153	379	328	290	260	235	215	198	183	170	160	150	141	134	127	121
8-#11	.0312	.156	416	360	317	283	256	234	215	199	185	173	162	153	145	137	130
10-#6	.0110	.136	286	252	225	203	185	170	158	146	137	129	121	115	109	103	95
10-#7	.0150	.140	312	274	244	220	200	183	169	157	147	138	130	123	116	111	105
10-#8	.0198	.147	342	298	264	237	215	197	182	168	157	147	138	131	124	117	112
10-#9	.0250	.152	376	326	288	258	234	214	196	182	170	159	149	141	133	126	120
10-#10	.0317	.156	419	363	319	285	258	236	217	200	186	174	164	154	146	138	132
10-#11	.0390	.162	466	401	352	314	282	257	236	218	203	190	178	167	158	150	142
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† Below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—22" x 22"

Bars*	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#10	.0105	.124	342	304	274	249	229	211	196	183	172	162	153	145	137	131	125
4-#11	.0129	.126	361	321	288	262	240	222	206	192	180	169	160	151	144	137	130
6-#9	.0124	.125	357	317	285	260	238	220	204	190	179	168	159	150	143	136	130
6-#10	.0158	.128	383	340	305	276	253	234	217	202	189	178	168	159	151	144	137
6-#11	.0193	.133	411	363	325	294	268	247	229	213	199	187	176	167	158	150	143
8-#8	.0131	.126	362	322	289	263	241	222	206	192	180	170	160	152	144	137	131
8-#9	.0165	.130	389	344	309	280	256	236	218	204	191	179	169	160	152	145	138
8-#10	.0210	.134	424	374	335	302	276	254	235	219	205	192	181	172	163	154	148
8-#11	.0258	.139	461	405	361	326	297	272	252	234	218	205	193	182	173	164	156
10-#7	.0124	.125	357	317	285	260	238	220	204	190	178	168	159	150	143	136	130
10-#8	.0163	.129	387	343	308	279	255	235	218	203	191	179	169	160	152	145	138
10-#9	.0206	.134	421	371	332	300	274	252	234	218	204	191	180	170	161	153	147
10-#10	.0263	.140	464	406	362	326	297	273	252	234	219	205	193	183	173	164	157
10-#11	.0323	.142	511	447	398	358	326	298	276	256	240	224	211	199	189	180	171
12-#6	.0109	.124	346	308	278	252	231	214	198	185	174	164	154	146	139	133	127
12-#7	.0149	.127	376	334	300	272	250	230	213	199	187	175	166	157	149	142	136
12-#8	.0196	.133	413	364	327	296	270	248	230	214	200	188	177	168	159	151	144
12-#9	.0248	.138	453	398	355	320	292	268	248	230	215	202	190	179	171	162	155
12-#10	.0315	.142	505	442	394	354	322	296	273	254	237	222	209	197	187	178	169
12-#11	.0387	.147	561	490	435	390	354	324	298	276	258	241	227	214	203	193	184
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—24" x 24"

Bars *	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#11	.0108	.111	411	370	337	308	285	264	247	232	218	206	195	185	176	168	161
6-#9	.0104	.111	407	367	333	305	282	262	244	229	216	204	193	183	175	167	160
6-#10	.0132	.114	433	389	353	323	298	276	257	241	226	214	202	192	183	174	167
6-#11	.0162	.116	461	413	374	342	315	292	272	254	239	226	214	203	193	184	176
8-#8	.0110	.113	412	370	335	306	282	262	244	228	214	203	192	182	173	165	158
8-#9	.0139	.115	439	394	357	326	300	279	260	243	229	216	204	194	184	176	168
8-#10	.0176	.117	474	424	384	351	323	299	278	261	245	231	218	207	197	188	180
8-#11	.0216	.120	511	456	412	376	345	320	297	278	261	246	232	220	210	200	191
10-#7	.0104	.111	407	366	333	305	282	262	244	229	216	204	193	183	175	166	159
10-#8	.0137	.114	437	392	356	326	300	278	259	243	228	216	204	194	184	176	168
10-#9	.0174	.117	471	422	382	348	322	297	277	259	244	230	217	206	196	187	179
10-#10	.0220	.120	514	459	414	378	347	321	299	279	262	247	234	221	210	201	192
10-#11	.0270	.123	561	500	450	410	376	348	323	301	283	266	252	238	227	216	206
12-#7	.0125	.113	426	383	348	319	294	272	254	238	224	212	200	190	181	173	165
12-#8	.0164	.116	463	415	376	334	316	293	273	256	240	226	214	204	194	185	176
12-#9	.0208	.119	503	459	407	371	341	316	294	274	258	243	230	218	207	198	189
12-#10	.0265	.123	555	494	445	406	372	344	320	298	280	263	249	236	224	214	204
12-#11	.0325	.124	611	544	490	446	409	378	351	328	307	289	273	258	246	234	224
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—26″ x 26″

Bars *	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#10	.0113	.103	487	441	404	372	345	322	301	283	267	253	240	228	218	208	199
6-#11	.0138	.106	515	466	425	391	362	337	315	296	279	264	250	238	227	217	207
8-#9	.0118	.104	493	446	408	376	348	324	303	285	269	255	242	230	219	210	201
8-#10	.0150	.106	528	478	435	401	371	345	323	303	286	270	256	244	232	222	212
8-#11	.0184	.109	565	510	465	426	394	366	342	320	302	285	270	257	245	234	224
10-#8	.0117	.104	491	445	406	374	347	323	302	284	268	254	241	229	218	209	200
10-#9	.0148	.106	525	475	434	398	369	343	321	301	284	269	255	242	231	221	211
10-#10	.0188	.109	568	511	466	427	396	367	343	322	303	286	272	258	246	235	225
10-#11	.0235	.112	615	553	503	461	425	395	368	345	325	306	290	276	262	251	240
12-#7	.0106	.103	480	435	398	367	340	317	297	279	263	249	236	225	215	206	197
12-#8	.0140	.106	517	467	426	393	363	338	316	297	280	265	251	239	228	218	208
12-#9	.0177	.108	557	504	459	421	389	362	338	317	299	282	268	255	243	232	222
12-#10	.0227	.111	609	548	498	456	422	392	366	343	323	305	289	274	261	249	239
12-#11	.0277	.114	665	597	542	495	457	424	395	370	348	329	311	295	281	268	256
14-#7	.0124	.104	499	452	414	380	352	328	307	289	272	258	244	232	222	212	203
14-#8	.0163	.107	542	490	447	410	380	353	330	310	292	276	262	249	237	227	217
14-#9	.0207	.110	589	530	482	442	409	380	355	332	313	296	280	266	254	242	232
14-#10	.0263	.114	650	584	530	484	446	414	386	362	340	320	304	288	274	262	251
14-#11	.0323	.114	714	640	581	531	490	455	424	397	373	352	334	317	301	287	275
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—28" x 28"

Bars *	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#11	.0119	.097	573	523	480	444	413	386	362	342	323	306	291	277	265	253	242
8-#9	.0102	.095	551	504	464	429	400	374	351	331	313	297	283	269	258	246	236
8-#10	.0130	.097	586	535	491	454	423	395	370	350	330	313	298	284	271	259	248
8-#11	.0160	.099	623	567	521	481	446	416	391	368	348	330	313	298	285	272	261
10-#8	.0101	.095	549	502	461	427	398	373	350	330	312	296	282	269	256	246	235
10-#9	.0128	.097	583	532	489	451	421	392	369	348	328	311	296	282	269	258	246
10-#10	.0162	.099	626	570	522	483	449	419	392	370	349	331	314	300	286	274	261
10-#11	.0199	.102	673	610	560	516	479	446	418	393	371	351	333	317	303	289	277
12-#8	.0121	.097	575	525	482	445	415	387	363	343	324	307	292	278	266	254	243
12-#9	.0153	.099	615	560	514	475	440	412	386	363	343	325	309	295	281	269	257
12-#10	.0195	.101	667	605	554	512	475	443	415	391	369	350	332	316	302	289	277
12-#11	.0238	.104	723	655	599	550	511	475	445	419	394	374	354	337	322	308	295
14-#7	.0107	.095	557	509	468	434	403	378	355	334	316	300	286	271	260	249	238
14-#8	.0129	.097	600	547	502	465	433	404	379	358	338	320	305	290	277	265	253
14-#9	.0178	.101	647	587	538	496	461	430	403	380	358	339	322	307	293	280	268
14-#10	.0227	.103	708	641	587	541	501	468	438	411	388	367	349	332	317	303	290
14-#11	.0278	.106	772	698	636	586	541	504	472	443	418	395	375	356	340	325	310
16-#7	.0122	.097	577	526	484	447	416	389	364	344	325	308	293	279	266	255	244
16-#8	.0162	.098	625	570	522	483	449	419	394	371	350	332	316	301	287	275	262
16-#9	.0204	.102	679	616	564	520	482	450	420	396	374	356	336	319	305	292	279
16-#10	.0258	.106	748	676	616	568	525	489	457	429	405	383	363	345	329	315	301
16-#11	.0317	.106	822	744	678	625	577	537	503	471	445	421	399	379	362	346	331
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—30″ x 30″

Bars*	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
8-#10	.0112	.087	649	597	553	514	481	452	426	404	383	364	347	332	317	305	293
8-#11	.0134	.089	686	630	583	542	506	475	447	423	401	381	363	347	332	319	306
10-#9	.0111	.087	646	595	550	513	480	451	425	402	381	363	346	331	316	304	292
10-#10	.0141	.089	689	632	585	544	509	476	449	424	402	382	364	348	333	320	307
10-#11	.0173	.091	736	675	623	578	540	506	476	450	426	405	386	368	352	337	324
12-#8	.0105	.087	638	587	544	505	474	445	419	397	376	358	341	326	312	300	288
12-#9	.0133	.089	678	623	576	535	500	469	442	418	396	376	359	343	328	315	302
12-#10	.0169	.091	730	669	617	574	535	502	472	446	423	402	382	365	349	334	321
12-#11	.0207	.092	786	720	664	616	575	539	506	479	454	430	410	392	373	358	343
14-#8	.0123	.088	663	610	565	525	490	460	435	410	389	370	353	337	323	309	297
14-#9	.0155	.089	710	651	604	560	524	491	463	437	415	394	376	359	343	330	315
14-#10	.0197	.091	771	706	653	605	565	530	499	471	447	425	404	385	369	353	339
14-#11	.0242	.093	835	764	705	648	609	570	536	505	479	455	432	412	395	378	362
16-#7	.0107	.087	640	590	545	507	475	446	420	398	378	359	342	327	313	300	289
16-#8	.0141	.089	688	632	585	544	508	476	449	424	402	382	364	348	333	320	306
16-#9	.0178	.091	742	680	627	582	544	510	480	454	430	408	388	370	355	340	326
16-#10	.0225	.093	811	742	685	635	591	554	521	492	465	442	420	401	383	367	352
16-#11	.0276	.095	885	809	744	689	641	600	564	531	503	477	454	433	414	396	378
18-#7	.0120	.088	659	606	560	521	487	458	431	408	386	368	350	335	321	307	295
18-#8	.0158	.089	713	655	606	563	526	494	465	439	416	396	377	361	345	331	318
18-#9	.0200	.092	774	707	653	606	566	530	498	470	445	424	403	385	367	353	338
18-#10	.0254	.094	852	779	719	664	620	580	545	514	486	462	439	419	400	383	368
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—12″ x 12″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#6	.0122	.233	125	101	85	73	65	58	52	48	44	30	26	22	19	17	15
4-#7	.0167	.235	135	109	92	79	69	62	56	51	47	34	30	27	25	22	20
4-#8	.0220	.241	148	119	100	86	75	67	60	55	50	38	34	31	28	26	24
4-#9	.0278	.256	161	128	106	91	80	70	63	58	53	43	38	35	32	29	27
4-#10	.0355	.285	178	138	113	96	83	73	65	59	54	48	44	39	36	33	31
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	†	†	†	†	†	†

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—14″ x 14″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#7	.0122	.200	170	142	121	106	95	85	77	71	65	61	47	41	35	31	27
4-#8	.0162	.208	183	152	130	112	100	90	81	74	68	64	53	48	43	39	35
4-#9	.0204	.216	196	162	137	119	105	94	85	78	72	67	59	53	49	45	41
4-#10	.0258	.223	213	174	147	128	113	101	91	83	76	71	66	60	55	50	47
4-#11	.0322	.235	232	187	157	136	119	106	96	87	80	74	69	65	61	56	52
6-#6	.0135	.206	174	145	124	108	95	86	78	71	66	61	49	44	38	33	29
6-#7	.0183	.212	190	157	133	116	103	92	84	76	70	65	56	50	46	42	39
6-#8	.0242	.220	208	170	145	125	111	99	90	82	75	70	64	58	53	49	45
6-#9	.0306	.233	228	185	156	136	118	105	95	87	80	76	68	64	60	55	51
6-#10	.0390	.245	254	203	170	147	128	115	103	93	85	79	74	69	65	61	57
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	†	†	†	†	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—16" x 16"

Bars*	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#8	.0124	.172	224	191	166	148	133	120	110	102	94	88	82	68	61	52	46
4-#9	.0156	.176	237	201	175	155	139	126	115	106	98	92	86	75	69	63	58
4-#10	.0198	.182	254	215	186	164	147	133	121	112	104	96	90	84	77	71	66
4-#11	.0245	.188	273	230	198	175	156	141	128	118	109	101	95	89	84	79	73
6-#6	.0103	.168	215	184	161	143	129	117	107	99	92	86	80	62	52	45	39
6-#7	.0141	.173	231	197	172	152	136	124	113	104	97	90	85	72	66	59	52
6-#8	.0185	.179	249	211	183	162	145	131	120	110	102	95	89	81	74	69	64
6-#9	.0235	.187	269	227	196	172	154	139	127	116	108	100	94	88	83	77	72
6-#10	.0297	.198	295	246	211	185	165	148	135	124	114	106	99	93	87	82	78
6-#11	.0365	.205	323	268	229	200	178	160	145	132	122	114	106	99	93	88	83
8-#6	.0137	.173	229	195	170	151	135	123	112	104	96	90	84	71	65	58	51
8-#7	.0188	.180	250	212	184	162	145	132	120	111	102	95	89	82	75	69	64
8-#8	.0247	.188	274	230	199	175	156	141	129	118	109	102	95	89	84	79	74
8-#9	.0312	.200	301	251	215	188	167	150	137	125	116	107	100	94	88	84	79
8-#10	.0397	.207	336	278	237	207	184	165	150	137	126	117	109	103	96	91	86
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	†	†	†	†

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—18" x 18"

Bars*	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#9	.0123	.153	283	245	217	194	176	160	148	137	127	119	112	105	100	85	74
4-#10	.0157	.156	300	260	229	204	185	168	155	143	133	125	117	110	104	95	88
4-#11	.0192	.161	319	275	241	215	194	177	162	150	139	130	122	115	109	103	98
6-#7	.0111	.151	277	240	213	191	173	158	145	135	125	117	110	104	98	78	68
6-#8	.0146	.154	295	256	225	202	182	167	153	142	132	124	116	109	104	92	85
6-#9	.0185	.159	315	272	239	213	192	175	161	149	139	130	122	114	108	103	95
6-#10	.0235	.166	341	292	256	228	205	186	171	158	146	137	128	121	114	108	102
6-#11	.0289	.175	369	314	273	247	217	197	180	166	154	143	134	126	119	113	107
8-#6	.0109	.150	275	239	212	190	172	157	145	134	125	117	110	104	98	76	67
8-#7	.0148	.155	296	256	226	202	183	167	153	142	132	124	116	109	103	93	86
8-#8	.0195	.161	320	276	242	216	194	177	163	150	140	131	122	115	109	103	98
8-#9	.0247	.167	347	298	260	231	208	189	173	160	149	138	130	122	115	109	104
8-#10	.0313	.178	382	324	282	249	223	202	184	170	158	147	137	129	122	116	109
8-#11	.0385	.183	419	354	307	270	242	219	200	184	170	158	148	139	131	124	118
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	†	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† Below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—20″ x 20″

Bars *	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#9	.0100	.131	334	295	265	240	219	201	187	174	163	153	144	137	130	123	96
4-#10	.0127	.135	351	309	276	250	228	210	194	180	169	158	149	141	134	127	114
4-#11	.0156	.138	370	325	290	262	238	219	202	188	176	165	155	147	139	132	126
6-#8	.0118	.134	346	305	273	247	225	207	192	178	167	157	148	140	132	126	111
6-#9	.0150	.137	366	322	287	259	236	217	201	187	174	164	154	146	138	132	123
6-#10	.0190	.142	392	343	305	275	250	229	212	197	184	172	162	153	145	138	131
6-#11	.0234	.146	420	366	325	292	265	243	224	207	194	181	171	161	153	145	138
8-#7	.0120	.134	347	306	274	248	226	208	192	179	168	157	148	140	133	126	111
8-#8	.0158	.138	371	326	291	262	239	220	203	189	176	165	156	147	140	133	126
8-#9	.0200	.142	398	349	310	279	254	233	215	200	186	175	164	155	147	140	133
8-#10	.0254	.148	433	377	334	300	272	249	229	213	198	186	175	165	156	148	141
8-#11	.0312	.155	470	407	358	321	290	265	243	225	210	196	184	174	164	156	148
10-#6	.0110	.133	340	300	269	243	222	204	189	176	165	155	146	138	131	125	104
10-#7	.0150	.137	366	322	287	259	236	217	201	187	174	164	154	146	138	132	123
10-#8	.0198	.142	396	346	308	278	252	232	214	198	186	174	164	154	146	139	132
10-#9	.0250	.147	430	375	332	298	271	248	228	212	198	185	174	164	156	148	141
10-#10	.0317	.155	473	410	361	323	292	266	245	227	211	198	186	175	165	157	149
10-#11	.0390	.159	520	448	394	352	318	290	266	246	229	214	200	189	179	169	161
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	†

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

† Concrete governs and safe load, N, is the same for 16,000 psi steel.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\quad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—22″ x 22″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#10	.0105	.120	408	364	329	300	274	255	237	222	208	196	186	176	167	159	152
4-#11	.0129	.123	427	380	343	312	286	264	246	229	215	203	191	182	173	165	157
6-#9	.0124	.123	423	376	340	309	284	262	244	227	214	201	190	180	171	163	155
6-#10	.0158	.125	449	399	359	326	300	276	256	240	224	211	200	189	180	171	163
6-#11	.0193	.130	477	421	378	343	314	289	268	250	234	220	207	196	186	177	169
8-#8	.0131	.123	428	381	344	314	287	265	246	230	216	203	192	182	173	165	157
8-#9	.0165	.126	455	404	363	331	302	279	259	242	226	214	201	191	181	173	164
8-#10	.0210	.131	490	433	388	352	322	296	274	256	239	225	212	200	191	181	173
8-#11	.0258	.134	527	464	415	375	342	314	291	271	253	238	224	212	201	191	182
10-#7	.0124	.123	423	376	340	309	284	262	244	227	214	201	190	180	171	163	155
10-#8	.0163	.126	453	403	362	329	302	278	258	240	226	212	201	190	181	172	164
10-#9	.0206	.130	487	431	386	350	320	295	274	255	238	224	212	200	190	181	173
10-#10	.0263	.135	530	466	417	377	344	316	293	272	255	240	226	213	202	192	183
10-#11	.0323	.140	577	505	450	406	370	339	314	291	272	255	240	227	215	204	195
12-#6	.0109	.121	412	367	331	302	278	256	238	223	209	197	186	177	168	160	153
12-#7	.0149	.125	442	393	354	322	295	272	253	236	221	208	197	186	177	169	161
12-#8	.0196	.130	479	424	380	345	315	290	269	251	235	221	208	197	187	178	170
12-#9	.0248	.134	519	457	410	370	338	311	288	268	251	236	222	210	200	189	181
12-#10	.0315	.140	571	501	446	403	366	336	310	288	270	253	238	225	213	202	193
12-#11	.0387	.144	627	548	487	438	398	365	336	312	292	273	257	242	230	218	208
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—24″ x 24″

Bars *	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
4-#11	.0108	.109	489	441	401	369	341	316	296	277	261	246	234	223	212	202	193
6-#9	.0104	.110	485	437	398	365	337	313	292	274	258	244	231	219	209	200	191
6-#10	.0132	.111	511	460	419	384	354	329	307	288	271	256	242	230	220	209	200
6-#11	.0162	.113	539	484	439	402	371	344	321	300	283	268	253	240	229	216	208
8-#8	.0110	.109	490	441	402	369	341	317	296	278	262	247	234	223	212	203	194
8-#9	.0139	.111	517	465	423	388	358	332	310	291	274	259	245	233	222	212	202
8-#10	.0176	.114	552	495	450	411	379	352	328	307	288	272	258	245	233	222	213
8-#11	.0216	.118	589	527	476	435	400	370	345	322	303	286	270	256	244	232	222
10-#7	.0104	.108	485	438	399	366	339	315	294	276	260	246	233	222	219	202	193
10-#8	.0137	.111	515	464	421	386	357	331	310	290	273	258	244	232	221	211	202
10-#9	.0174	.114	549	492	447	409	377	350	326	306	287	271	256	244	232	221	212
10-#10	.0220	.118	592	530	480	437	402	372	347	324	304	287	272	258	245	234	224
10-#11	.0270	.120	639	570	515	470	431	399	371	347	326	307	290	275	262	250	238
12-#7	.0125	.111	504	454	412	378	349	324	302	284	267	252	239	227	216	206	198
12-#8	.0164	.113	541	486	441	405	372	346	323	302	284	269	254	241	230	219	210
12-#9	.0208	.118	581	520	470	430	395	366	341	319	299	282	267	254	241	230	219
12-#10	.0265	.120	633	565	510	465	428	396	368	344	323	304	288	273	260	248	236
12-#11	.0325	.123	689	612	552	502	460	425	395	369	346	326	307	291	277	264	252
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—26″ x 26″

Bars *	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#10	.0113	.101	578	525	480	443	412	384	360	339	320	303	288	274	261	250	239
6-#11	.0138	.103	606	550	503	464	430	400	375	352	332	315	299	285	272	259	248
8-#9	.0118	.101	584	530	485	448	416	388	364	342	324	306	290	277	264	253	242
8-#10	.0150	.104	619	560	511	472	437	407	381	358	338	320	303	289	276	264	252
8-#11	.0184	.107	656	594	540	496	460	428	400	376	354	334	317	302	287	274	262
10-#8	.0117	.101	582	529	484	446	414	387	363	341	322	305	290	276	263	252	241
10-#9	.0148	.103	616	558	511	471	436	407	381	358	338	320	304	289	276	264	252
10-#10	.0188	.107	659	595	542	498	461	429	401	379	355	336	318	303	288	275	264
10-#11	.0235	.109	706	637	580	532	491	457	427	400	377	356	338	321	306	292	279
12-#7	.0106	.100	571	520	476	440	409	381	357	336	318	301	286	272	260	249	238
12-#8	.0140	.103	608	551	505	465	431	401	376	353	334	316	300	286	272	260	249
12-#9	.0177	.106	648	586	534	492	455	424	396	372	351	332	314	299	285	272	261
12-#10	.0227	.109	700	631	575	528	488	454	424	397	374	354	335	319	304	290	277
12-#11	.0277	.111	756	680	619	566	524	486	454	426	401	378	358	340	324	310	296
14-#7	.0124	.102	590	535	490	452	420	391	366	344	325	308	292	278	265	254	243
14-#8	.0163	.104	633	574	524	482	447	416	390	366	345	327	310	295	281	269	258
14-#9	.0207	.108	680	615	560	513	475	441	413	387	365	345	327	311	296	283	271
14-#10	.0263	.111	741	666	606	556	514	476	445	418	393	371	352	334	318	303	290
14-#11	.0323	.115	805	722	655	599	551	511	476	446	419	396	375	355	338	323	309
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—28″ x 28″

Bars*	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#11	.0119	.094	679	620	571	529	494	461	434	410	387	368	350	334	319	305	293
8-#9	.0102	.092	657	601	555	515	481	450	424	400	379	360	342	327	312	300	287
8-#10	.0130	.095	692	631	581	539	501	469	440	415	393	373	355	338	323	310	296
8-#11	.0160	.097	729	665	610	565	525	490	460	431	410	389	370	352	337	322	308
10-#8	.0101	.092	655	600	554	514	480	449	422	398	378	358	341	326	311	298	286
10-#9	.0128	.095	689	629	579	535	499	467	438	414	391	371	353	336	322	308	294
10-#10	.0162	.097	732	668	613	566	528	493	462	436	412	391	372	354	338	324	309
10-#11	.0199	.100	779	709	650	599	556	519	486	458	433	410	390	371	354	339	324
12-#8	.0121	.094	681	622	574	531	495	464	435	411	389	369	351	335	320	306	294
12-#9	.0153	.096	721	659	605	560	521	487	458	432	408	387	368	351	335	321	307
12-#10	.0195	.100	773	702	644	595	552	515	483	455	430	407	387	368	351	336	322
12-#11	.0238	.101	829	753	690	636	591	551	516	486	459	435	412	393	375	359	344
14-#7	.0107	.092	663	607	560	520	485	454	427	403	382	363	345	330	315	302	290
14-#8	.0129	.095	706	645	594	549	511	479	450	424	401	381	362	345	330	316	302
14-#9	.0178	.100	753	685	627	579	537	501	470	443	419	396	376	357	342	327	314
14-#10	.0227	.101	814	739	676	625	580	541	507	477	451	427	405	386	368	352	338
14-#11	.0278	.104	878	795	728	669	620	576	540	509	479	454	430	409	390	374	358
16-#7	.0122	.094	683	624	575	533	496	465	436	411	390	370	352	336	321	307	295
16-#8	.0162	.096	731	668	614	569	529	494	464	437	414	392	373	356	340	325	311
16-#9	.0204	.100	785	714	654	604	560	524	491	462	436	413	392	374	357	341	327
16-#10	.0258	.103	854	774	708	652	604	564	528	496	468	443	420	400	382	365	350
16-#11	.0317	.105	928	840	765	705	651	606	567	531	503	475	450	428	408	390	374
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

SQUARE TIED COLUMNS—Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 0.8 \times 20{,}000 = 16{,}000$ psi

COLUMN SIZE—30″ x 30″

Bars *	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
8-#10	.0112	.086	771	710	657	614	574	540	509	481	457	435	415	396	380	365	350
8-#11	.0134	.087	808	744	688	640	600	563	530	502	476	453	432	413	395	370	364
10-#9	.0111	.086	768	708	655	611	571	537	506	479	455	433	413	395	378	362	348
10-#10	.0141	.087	811	747	691	644	602	565	534	505	479	455	434	415	397	381	366
10-#11	.0173	.089	858	787	729	677	633	594	559	528	501	476	454	434	415	398	382
12-#8	.0105	.086	760	700	649	605	565	531	501	474	450	428	409	391	374	359	345
12-#9	.0133	.087	800	736	681	634	594	557	526	497	472	448	428	409	391	376	361
12-#10	.0169	.089	852	782	725	673	629	590	555	525	497	473	451	431	412	395	380
12-#11	.0207	.091	908	831	767	712	665	624	587	555	526	500	475	454	435	415	400
14-#8	.0123	.087	785	723	669	622	583	547	515	489	464	440	420	401	384	369	354
14-#9	.0155	.087	832	765	709	660	617	580	546	516	490	466	445	425	406	390	375
14-#10	.0197	.091	893	817	755	701	655	614	578	546	517	491	468	445	427	408	393
14-#11	.0242	.091	957	876	810	751	701	658	620	585	555	526	501	479	458	438	421
16-#7	.0107	.086	762	702	650	606	566	533	503	475	452	430	410	392	375	360	346
16-#8	.0141	.087	810	745	690	642	601	565	532	504	478	454	433	414	396	380	366
16-#9	.0178	.089	864	794	734	682	637	598	563	531	504	479	457	437	418	401	384
16-#10	.0225	.091	933	855	790	732	684	641	604	570	540	514	489	466	446	427	411
16-#11	.0276	.093	1007	920	850	786	734	687	646	610	576	549	521	497	476	456	437
18-#7	.0120	.087	781	719	665	619	580	544	513	486	460	438	418	399	382	367	352
18-#8	.0158	.087	835	769	711	661	620	582	549	519	493	468	446	426	408	392	377
18-#9	.0200	.091	896	821	758	704	656	616	580	547	519	493	470	448	428	410	394
18-#10	.0254	.092	974	890	822	764	712	667	627	592	561	534	507	484	462	444	425
For $f_s = 0.8 \times 16{,}000 =$ 12,800 psi multiply by				.930	.940	.940	.945	.955	.955	.955	.955	.955	.965	.965	.965	.965	.965

* One-half of the bars are to be placed in each of the two faces of the column that are perpendicular to the plane of bending.

ECCENTRICALLY LOADED CONCRETE COLUMNS
SECTION II—SPIRALLY REINFORCED SQUARE COLUMNS

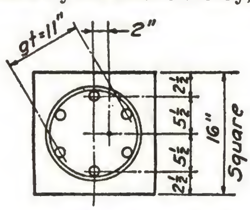
This second section covers eccentric loads on spirally reinforced square concrete columns and parallels exactly the previous section, so the explanation on pages 275 to 280, inclusive, should be read before going on with the following description.

The necessary amount of spiral reinforcement can be taken from the tables for axially loaded spirally reinforced square concrete columns on pages 264 to 267, inclusive. The vertical bars are spaced uniformly around a ring just inside of and in contact with the spiral.

While the scope of these tables is sufficient for most purposes, some illustrative examples are shown for those who wish to design beyond their range or to see how they were prepared.

Example—For the table on page 300, verify the value $N = 177$ with an eccentricity of 2 in. for a 16 in. spirally reinforced square column of 3000 psi concrete reinforced with 6-#8 bars, using $f_s = 20{,}000$ psi, $f_c = 675$ psi, $n = 10$.

Since the point of application is well within the middle third of the 16 x 16 section, the load probably acts within the kern of the transformed section,* producing compression over the entire area. Solve by the elastic theory, using the transformed section:—



6-#8 Bars = 4.74 sq in. ×n = 42.7 sq in.

Section	Area of Transformed Section	Capacity (Concentric Axial Loading)	Moment of Inertia
Concrete	$16 \times 16 = 256.0$	$256 @ 675 = 172{,}800$	$256 \times \frac{16^2}{12} = 5460$
Bars	$9 \times 6 \times 0.79 = 42.7$	$6 \times 0.79 @ 20{,}000 = 94{,}800$	$\dagger\, \frac{42.7 \times (5.5)^2}{2} = 646$
	298.7 sq in.	267,600 lb	6106 in.^{4}

$$\text{Unit direct stress} = \frac{N}{A} = \frac{177{,}000}{298.7} = 593 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = \frac{177{,}000 \times 2 \times 8}{6106} = 464 \text{ psi}$$

1057 psi Max Comp
129 psi Min Comp

Neutral axis is $\frac{129 \times 16}{928} = 2.23$ in. outside column side; $f_s = 10 \times \frac{13.5 + 2.23}{18.23} \times 1057 = 9120$ psi Comp.

The tabulated value, $N = 177$ kips, is verified by the method outlined on page 276, Ex. I, viz. $\frac{f_a}{F_a} + \frac{f_b}{F_b} \lessgtr 1$; $\frac{177{,}000}{267{,}600} + \frac{464}{1350} = 0.660 + 0.343 = 1.003$

By the method of the 1951 Code (see page 277):—

$R^2 = \frac{6106}{298.7} = 20.4$ $\qquad D = \frac{16 \times 16}{2 \times 20.4} = 6.27$

$f_a = \frac{267{,}600}{298.7} = 896$ $\qquad C = \frac{896}{1350} = 0.663$

* See page 93.

† The transformed steel area is assumed to be a ring with a mean radius r_m of 5.5 in. $I = \frac{\pi}{64}(D_1^4 - D_2^4) = \frac{Ar_m^2}{2}$

ECCENTRICALLY LOADED SPIRALLY REINFORCED SQUARE COLUMNS

$$B/t = CD/t = 0.663 \times \frac{6.27}{16} = 0.260 \quad \text{(or from Table on page 300, } \frac{CD}{t} = 0.258\text{)}$$

$$f_p = 896 \frac{\left(1 + 6.27 \times \frac{2}{16}\right)}{\left(1 + \frac{0.663 \times 6.27 \times 2}{16}\right)} = 1055 \text{ psi allowable as compared with 1057 psi}$$

actual stress as computed above.

The simplified method illustrated in Ex. I—Second Solution on page 278 can also be applied here with some saving of time:—

$$D = \frac{1 + (n-1)p_g}{\frac{1}{6} + 0.25(n-1)p_g{}^2}, \text{ where } p_g = 6 \times \frac{0.79}{256} = 0.01852 \text{ and } g = \frac{11}{16} = 0.688, \text{ so that}$$

$$D = \frac{1 + 0.167}{0.167 + 0.25 \times 0.167 \times 0.688^2} = 6.25, \text{ and } \frac{B}{t} = \frac{CD}{t} = 0.663 \times \frac{6.25}{16} = 0.259, \text{ and}$$

$$N = \frac{268{,}000}{1 + 0.259 \times 2} = 177 \text{ kips}$$

Example—With the same data as in the previous example, show that when the eccentricity is increased to 6 in., the allowable eccentric load, N, is reduced to 105 kips as given in the table on page 300.

As explained on page 279, when the eccentricity is less than two-thirds the width of the column, the method of an uncracked section can be applied under the ACI Code,* and using the values established in the previous example:—

$$\text{Unit direct stress} = \frac{N}{A} = \frac{105{,}000}{298.7} = 353 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = 105{,}000 \times 6 \times \frac{8}{6106} = \underline{826} \text{ psi}$$

1179 psi Max Comp

473 psi Max Tens

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} = \frac{105{,}000}{267{,}600} + \frac{826}{1350} = 0.392 + 0.611 = 1.003$$

Using the shorter method described on page 278:—

$$N = \frac{268{,}000}{(1 + 0.259 \times 6)} = 105 \text{ kips}$$

---

* When tension in the concrete is neglected and when the vertical steel is considered as placed in a ring, the stress prism for the steel becomes a sloping portion of a hollow cylinder whose properties are somewhat involved trigonometrically. (See Sutherland and Reese "Reinforced Concrete Design," John Wiley & Sons, Inc., 1943, pages 124-125.) A cut-and-try method is about the only satisfactory approach to the cubic and trigonometric equations involved. Since the variation in the maximum compression as computed on page 280 was about 10 per cent and since often the data are not known any more precisely than this and since an accurate determination is too time-consuming for ordinary use, it is felt that any further development of this problem more properly belongs in a textbook than in a handbook.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—14″ x 14″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#5	.0095	.260	169	134	111	95	83	73	66	60	55	50	27	23	20	18	16
8-#5	.0126	.273	182	143	118	100	87	77	69	62	57	53	32	29	25	22	20
10-#5	.0158	.290	194	151	123	104	90	79	71	64	59	54	35	32	29	26	23
6-#6	.0135	.277	185	145	119	101	88	78	71	63	58	53	33	30	26	23	21
7-#6	.0157	.288	194	151	123	105	90	80	71	64	59	54	35	32	29	26	23
8-#6	.0180	.298	202	156	126	107	92	81	73	66	60	55	37	33	30	28	25
9-#6	.0202	.306	211	161	131	110	95	84	75	67	61	56	38	35	32	29	27
6-#7	.0184	.300	204	157	128	107	93	82	73	66	60	55	37	33	30	28	26
7-#7	.0214	.310	216	165	133	112	97	85	76	68	62	57	39	35	32	30	27
8-#7	.0254	.328	228	172	138	114	99	87	77	69	63	58	42	38	34	31	29
6-#8	.0252	.327	227	171	137	115	99	86	77	69	63	58	41	37	34	31	29
7-#8	.0282	.337	243	182	145	121	104	91	80	72	66	60	43	39	36	33	31
8-#8	.0322	.349	258	191	152	126	108	94	84	75	68	62	47	42	38	35	33
6-#9	.0306	.344	252	188	149	124	106	93	82	74	67	62	45	41	37	34	32
7-#9	.0357	.359	272	200	159	132	112	98	86	77	70	64	49	44	40	37	34
6-#10	.0390	.375	284	206	162	134	114	99	88	79	71	65	51	46	42	39	36
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	†	†	†	†	†

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—15″ x 15″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0117	.253	205	164	136	117	102	90	81	74	68	63	58	35	31	27	24
9-#6	.0176	.277	231	181	148	126	109	97	87	79	72	66	61	42	38	35	32
6-#7	.0160	.271	224	176	145	124	107	95	85	77	71	65	60	40	36	33	29
7-#7	.0187	.282	236	184	151	128	111	98	88	79	72	67	62	43	39	36	33
8#-7	.0213	.291	248	192	157	132	114	101	90	82	74	68	63	45	41	38	35
9-#7	.0240	.301	260	200	162	137	118	104	93	84	76	70	65	47	43	40	37
6-#8	.0211	.290	247	191	156	132	114	101	90	82	74	68	63	45	41	38	35
7-#8	.0246	.303	263	202	164	138	119	104	93	84	77	70	65	47	43	40	37
8-#8	.0281	.317	278	211	170	142	122	108	96	86	79	72	67	51	46	43	39
6-#9	.0267	.311	272	207	168	141	121	106	95	86	78	72	66	49	45	41	38
7-#9	.0311	.327	292	220	176	147	126	111	98	89	81	74	68	53	48	45	41
8-#9	.0355	.341	312	232	185	154	132	115	102	92	84	77	71	56	51	47	44
6-#10	.0339	.336	304	228	182	151	130	113	101	91	82	76	70	55	50	46	43
6-#11	.0416	.361	339	249	197	162	139	121	107	96	87	80	74	61	56	51	48
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	†	†	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—16″ x 16″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0103	.229	226	184	155	134	118	105	96	87	80	74	69	42	36	31	26
9-#6	.0155	.249	252	202	168	144	126	112	101	92	87	78	72	49	45	41	37
6-#7	.0140	.244	245	198	165	142	124	111	100	91	83	77	71	48	43	39	36
7-#7	.0164	.251	257	206	172	147	129	114	103	94	86	79	73	51	46	42	38
9-#7	.0210	.266	281	222	184	156	137	121	108	98	90	83	77	56	51	47	43
11-#7	.0258	.281	305	238	196	166	144	127	114	103	94	85	80	61	56	51	47
6-#8	.0185	.258	268	213	177	152	132	117	105	96	88	81	75	53	48	44	40
7-#8	.0216	.268	284	224	185	158	137	123	109	99	91	84	77	57	52	47	43
8-#8	.0246	.276	299	234	192	163	142	125	112	102	93	86	79	60	55	50	46
9-#8	.0278	.286	315	244	201	170	147	130	116	105	96	88	82	63	58	53	49
6-#9	.0234	.274	293	230	189	161	140	124	111	101	92	85	79	59	53	49	45
7-#9	.0273	.285	313	244	200	169	147	129	116	105	96	89	82	63	57	53	49
8-#9	.0312	.297	333	257	210	177	152	134	120	109	99	91	86	67	61	56	52
6-#10	.0298	.294	325	250	205	173	149	132	118	107	97	89	83	65	59	55	50
7-#10	.0347	.307	351	268	218	183	158	139	124	112	102	94	86	70	64	59	55
6-#11	.0366	.311	360	275	222	186	161	141	126	113	103	95	88	72	66	61	56
7-#11	.0426	.329	391	294	235	197	169	148	132	119	108	99	91	78	71	66	61
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	†	†	†	†

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—17″ x 17″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0125	.223	267	218	185	160	141	126	114	104	96	89	83	77	51	44	39
9-#7	.0187	.245	303	243	203	174	153	136	123	112	102	94	88	82	60	55	51
11-#7	.0228	.258	327	260	216	184	161	143	128	116	107	98	91	85	65	60	55
6-#8	.0164	.237	290	234	197	169	149	133	120	109	100	93	86	80	57	52	48
8-#8	.0219	.254	321	255	213	182	159	141	127	116	106	98	91	85	63	59	54
10-#8	.0273	.271	353	278	229	195	169	150	134	122	111	103	95	89	71	65	60
6-#9	.0208	.251	315	252	210	180	157	140	126	114	105	97	90	84	62	57	53
8-#9	.0277	.272	355	279	230	195	170	150	135	122	112	103	95	89	71	65	60
10-#9	.0346	.291	395	306	250	211	182	161	144	130	119	109	101	94	79	73	67
6-#10	.0264	.268	347	274	226	192	167	148	133	121	110	102	94	88	69	63	58
7-#10	.0308	.281	373	291	239	202	176	155	139	126	115	106	98	91	74	68	63
8-#10	.0352	.292	398	308	251	212	183	162	145	131	119	110	101	94	79	73	67
6-#11	.0324	.285	382	297	243	206	178	158	141	128	116	107	99	92	76	70	65
7-#11	.0378	.300	413	318	258	217	188	165	147	133	122	112	103	96	82	76	70
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	.985	†	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—18" x 18"

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0111	.207	291	241	206	180	159	143	130	119	110	102	95	89	84	52	45
9-#7	.0167	.223	327	267	226	196	173	154	140	128	118	109	101	95	89	64	59
11-#7	.0204	.234	351	286	239	206	182	162	146	133	123	113	105	99	92	70	64
6-#8	.0146	.219	314	258	218	190	168	150	136	123	114	106	99	92	87	61	56
7-#8	.0171	.224	330	270	228	198	174	156	141	129	119	109	102	95	89	65	59
8-#8	.0195	.232	345	281	236	204	179	160	145	132	121	112	104	97	91	68	63
9-#8	.0220	.240	361	292	244	210	184	164	148	135	124	114	106	100	93	72	66
11-#8	.0268	.252	393	314	261	224	196	174	157	142	130	120	112	104	98	78	73
6-#9	.0185	.230	339	276	232	201	177	158	143	130	120	110	103	96	91	67	62
7-#9	.0216	.238	359	290	243	210	184	164	148	135	124	115	106	99	93	72	66
8-#9	.0247	.247	379	304	254	218	190	170	153	139	127	118	109	102	96	76	70
9-#9	.0278	.254	399	319	264	226	198	176	158	144	132	121	113	105	99	80	74
10-#9	.0308	.263	419	332	275	235	204	181	163	148	135	125	116	108	101	84	78
6-#10	.0235	.243	371	299	249	214	188	168	151	137	126	116	108	101	95	74	69
7-#10	.0274	.251	397	318	265	227	198	176	158	144	132	122	113	106	99	79	73
8-#10	.0314	.265	422	334	276	236	205	182	163	148	135	125	116	108	101	84	78
9-#10	.0353	.275	448	353	290	246	214	189	170	154	141	129	120	112	105	89	83
6-#11	.0289	.256	406	323	269	230	201	178	160	145	133	123	114	106	100	81	75
7-#11	.0338	.271	437	344	284	241	210	186	166	151	138	127	118	110	103	87	81
8-#11	.0385	.283	469	366	300	254	220	195	174	158	144	132	123	114	107	93	87
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—19″ x 19″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0131	.200	339	282	242	212	188	169	154	141	130	121	113	106	100	70	65
7-#8	.0153	.206	355	294	251	219	194	175	159	145	134	124	116	109	102	75	69
8-#8	.0175	.212	370	305	260	226	200	180	163	149	137	127	118	111	104	79	73
9-#8	.0197	.217	386	317	269	234	206	185	168	153	141	131	122	114	107	82	76
10-#8	.0219	.223	402	328	278	241	212	190	172	157	144	134	124	116	109	86	80
11-#8	.0241	.229	418	340	287	248	218	195	176	160	148	136	127	119	111	90	83
6-#9	.0166	.209	364	301	257	224	198	178	161	148	136	126	118	110	104	77	71
7-#9	.0194	.217	384	316	268	233	206	184	167	152	140	130	121	113	107	82	76
8-#9	.0221	.224	404	330	279	241	213	190	172	157	145	134	125	116	109	87	80
9-#9	.0249	.231	424	344	290	250	220	197	178	162	149	138	128	120	112	91	85
10-#9	.0277	.238	444	358	301	259	227	203	183	166	153	141	131	123	115	96	89
11-#9	.0305	.244	464	372	312	268	235	209	188	171	157	145	135	126	118	100	93
6-#10	.0211	.221	396	324	275	238	210	188	170	155	143	132	123	115	108	85	78
7-#10	.0246	.230	422	343	289	250	220	196	177	162	148	137	128	120	112	91	84
8-#10	.0281	.239	447	360	302	260	228	204	184	167	153	142	132	123	115	96	89
9-#10	.0316	.247	473	379	316	271	238	212	190	173	159	147	136	127	119	102	95
10-#10	.0352	.255	498	406	330	282	246	219	197	179	164	151	140	131	123	108	100
6-#11	.0259	.234	431	349	294	253	223	199	179	163	150	139	129	120	113	93	86
7-#11	.0302	.244	462	371	310	267	234	208	187	170	156	144	134	125	118	100	93
8-#11	.0346	.254	494	394	327	280	245	218	196	178	163	150	139	130	122	107	99
9-#11	.0388	.263	525	416	344	293	256	227	204	185	169	156	145	135	126	114	106
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $f_s = 20,000$ psi

COLUMN SIZE—20″ x 20″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0118	.185	365	308	266	235	210	190	173	159	147	137	128	120	113	107	70
7-#8	.0138	.191	381	320	276	242	216	195	178	163	151	140	131	123	116	109	78
8-#8	.0158	.197	396	331	284	249	221	199	182	167	154	145	134	125	118	111	83
9-#8	.0178	.201	412	343	294	257	229	206	187	171	158	147	137	128	121	114	87
10-#8	.0198	.207	428	356	304	265	235	211	191	175	161	150	140	131	123	116	91
11-#8	.0217	.211	444	368	313	273	241	216	196	179	165	153	143	134	126	119	96
12-#8	.0237	.216	460	378	322	279	247	221	200	183	169	157	146	137	128	121	98
6-#9	.0150	.195	390	327	281	247	220	198	180	165	153	142	132	124	117	110	80
7-#9	.0175	.200	410	342	293	256	228	205	187	171	158	147	137	128	121	114	86
8-#9	.0200	.207	430	356	305	266	235	212	192	176	162	151	140	131	124	117	92
9-#9	.0225	.213	450	372	316	275	243	218	197	181	167	154	144	135	127	119	96
10-#9	.0250	.220	470	385	327	283	250	223	203	185	171	157	147	138	129	122	100
11-#9	.0275	.224	490	401	338	293	258	232	209	192	176	163	151	142	133	126	104
6-#10	.0191	.204	422	351	301	262	233	210	191	174	160	149	139	130	122	116	89
7-#10	.0222	.212	448	370	316	275	242	218	198	181	167	154	144	135	127	119	95
8-#10	.0254	.221	473	388	327	285	252	225	204	186	171	158	147	138	129	122	102
9-#10	.0286	.226	499	407	344	298	262	234	212	194	178	165	153	143	135	127	106
10-#10	.0317	.232	524	427	360	309	272	243	219	200	184	170	158	148	139	131	112
6-#11	.0234	.215	457	376	320	278	246	220	200	182	168	156	145	136	124	120	97
7-#11	.0273	.224	488	399	337	293	258	231	208	190	175	162	151	141	133	125	103
8-#11	.0312	.231	520	423	356	307	270	242	218	199	183	169	157	147	138	130	110
9-#11	.0351	.240	551	444	372	320	281	250	226	206	189	174	162	152	142	134	116
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below horizontal line concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—21″ x 21″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0108	.174	393	334	291	258	232	210	192	177	164	153	143	135	127	120	114
7-#8	.0125	.179	409	347	301	266	238	216	197	182	168	157	147	138	130	123	117
8-#8	.0143	.184	424	358	310	273	244	221	201	185	172	160	149	140	132	125	118
9-#8	.0161	.188	440	370	320	281	251	227	207	190	176	163	153	143	135	128	121
10-#8	.0179	.192	456	383	329	289	258	233	212	194	180	167	156	147	138	130	124
11-#8	.0197	.197	472	394	338	296	264	238	216	198	183	170	159	149	140	132	126
12-#8	.0215	.201	488	406	348	304	270	243	221	203	187	174	162	152	143	135	128
6-#9	.0136	.182	418	354	306	270	242	219	200	184	170	159	148	139	131	124	118
7-#9	.0159	.188	438	369	318	280	250	226	206	189	175	163	152	143	134	127	121
8-#9	.0181	.193	458	384	331	290	258	233	212	195	180	168	156	147	138	131	124
9-#9	.0204	.198	478	399	342	300	267	240	218	200	185	172	160	150	142	134	127
10-#9	.0227	.204	498	413	353	309	274	246	224	205	189	175	164	153	144	137	129
11-#9	.0249	.209	518	428	365	318	282	253	230	210	194	180	168	157	148	139	132
12-#9	.0272	.214	538	442	376	327	290	260	235	215	198	184	171	160	151	142	135
6-#10	.0173	.191	450	377	325	286	255	230	210	193	178	165	155	145	137	129	122
7-#10	.0202	.198	476	397	341	298	268	239	217	200	184	171	160	150	141	133	126
8-#10	.0230	.204	501	416	356	311	276	248	225	206	190	177	165	154	145	137	130
9-#10	.0259	.211	527	435	370	322	286	256	232	213	196	182	169	159	149	141	133
10-#10	.0288	.218	552	453	384	334	295	264	239	219	201	186	174	162	153	144	136
6-#11	.0212	.201	485	403	346	302	268	242	220	201	186	172	161	151	142	134	128
7-#11	.0248	.209	516	427	364	317	281	252	229	209	193	179	167	156	147	139	132
8-#11	.0283	.217	548	450	382	332	293	263	238	218	200	185	173	162	152	143	136
9-#11	.0318	.223	579	473	400	347	306	274	248	226	208	192	179	168	158	148	140
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—22″ x 22″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0114	.167	438	376	329	292	263	239	219	203	188	176	164	155	147	138	132
8-#8	.0131	.172	453	387	337	299	269	244	223	206	191	178	167	157	148	140	133
9-#8	.0147	.176	469	399	347	307	275	250	228	211	195	182	170	160	151	143	136
10-#8	.0163	.179	485	412	357	316	283	255	234	215	199	186	174	163	154	146	139
11-#8	.0179	.183	501	425	367	324	290	262	239	220	204	190	177	167	157	148	141
12-#8	.0196	.187	517	435	377	330	295	267	243	224	207	192	180	169	160	150	143
6-#9	.0124	.170	447	382	335	297	267	242	222	204	190	177	166	156	147	140	132
7-#9	.0145	.176	467	400	346	307	275	249	228	210	195	181	170	159	150	142	135
8-#9	.0165	.179	487	415	358	318	284	258	236	216	200	187	175	164	155	147	139
9-#9	.0186	.184	507	426	370	327	292	263	240	222	205	190	178	168	158	149	142
10-#9	.0206	.190	527	442	381	336	300	270	247	227	210	194	182	170	160	152	144
11-#9	.0227	.194	547	460	396	345	308	277	252	232	213	200	186	174	164	155	147
12-#9	.0248	.199	567	474	403	355	315	283	258	237	219	202	189	178	167	158	150
6-#10	.0157	.178	479	407	353	313	280	253	233	213	198	184	172	162	153	145	137
7-#10	.0183	.184	505	426	370	325	290	263	240	222	205	190	178	167	157	149	142
8-#10	.0210	.191	530	446	383	338	302	272	248	227	211	195	183	170	161	152	145
9-#10	.0236	.196	556	465	397	352	312	280	257	235	217	201	188	176	166	157	148
10-#10	.0262	.202	581	485	413	362	322	290	264	242	223	207	193	181	170	161	153
11-#10	.0289	.207	606	500	430	374	332	298	270	247	228	212	197	186	174	164	155
6-#11	.0193	.186	514	433	375	330	295	267	244	223	207	193	180	169	160	151	143
7-#11	.0226	.194	545	457	391	345	309	277	252	232	213	200	187	175	165	155	147
8-#11	.0258	.201	577	480	412	360	320	287	261	240	222	206	192	180	169	160	151
9-#11	.0290	.207	608	500	430	374	332	298	270	247	228	212	197	186	174	164	155
10-#11	.0323	.212	639	530	450	390	347	310	282	258	237	220	205	192	181	170	161
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load ni Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—23″ x 23″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0105	.155	468	405	357	319	289	264	242	224	209	195	184	173	163	155	148
8-#8	.0120	.157	483	418	367	328	296	270	249	230	214	200	188	177	167	159	151
9-#8	.0134	.163	499	429	376	335	302	275	252	233	216	202	190	179	169	160	152
10-#8	.0150	.167	515	442	386	343	309	280	257	237	220	205	193	182	172	162	154
11-#8	.0164	.169	531	454	397	352	316	287	263	243	225	210	197	186	175	166	157
12-#8	.0179	.172	547	467	407	361	324	294	269	248	230	215	201	189	178	169	160
6-#9	.0114	.157	477	412	363	324	293	267	245	227	211	197	185	175	165	156	149
7-#9	.0132	.162	497	428	375	334	301	274	252	233	216	202	189	178	169	160	152
8-#9	.0152	.167	517	443	387	344	310	281	258	238	221	206	193	182	172	163	155
9-#9	.0170	.170	537	459	401	356	319	290	266	245	227	212	199	187	177	167	159
10-#9	.0189	.173	557	475	414	367	329	298	273	252	233	218	204	191	181	171	162
11-#9	.0208	.177	577	490	426	377	337	306	279	258	239	222	208	196	185	175	166
12-#9	.0227	.179	597	506	440	388	348	315	288	265	245	228	214	201	189	179	170
6-#10	.0144	.166	509	437	382	340	306	278	255	235	219	204	191	180	170	161	153
7-#10	.0168	.169	535	458	400	355	319	290	266	245	227	212	199	187	177	167	159
8-#10	.0192	.173	560	477	416	368	331	300	275	253	235	219	205	193	182	172	163
9-#10	.0217	.178	586	498	432	382	342	310	283	261	241	225	210	198	187	177	168
10-#10	.0240	.183	611	516	447	395	352	319	291	268	248	231	216	202	191	181	172
11-#10	.0264	.191	636	532	460	404	358	325	296	272	251	234	218	205	193	182	173
6-#11	.0177	.171	544	464	405	359	323	293	268	248	230	214	201	189	178	169	160
7-#11	.0207	.176	575	489	425	376	337	306	280	257	239	222	208	196	184	175	166
8-#11	.0237	.181	607	514	446	393	352	319	291	267	248	231	216	203	192	181	172
9-#11	.0266	.187	638	537	464	408	365	330	300	276	255	237	222	209	196	186	177
10-#11	.0295	.194	669	559	482	423	377	340	309	284	262	243	227	213	201	190	180
11-#11	.0324	.199	700	584	500	438	389	351	318	292	270	250	234	219	206	195	185
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—24″ x 24″

Bars	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#7	.0104	.149	509	443	392	352	319	292	269	251	233	218	205	193	182	174	165
12-#7	.0125	.153	533	462	408	366	332	302	278	248	238	224	211	199	188	175	170
8-#8	.0110	.150	515	448	396	355	322	294	271	251	234	219	206	194	184	174	166
10-#8	.0137	.157	547	474	416	372	336	306	282	261	243	227	213	201	190	180	171
12-#8	.0164	.162	579	499	438	390	351	320	294	272	252	236	221	208	197	187	178
6-#9	.0104	.149	509	443	392	352	319	292	269	250	233	218	205	193	182	174	165
8-#9	.0139	.158	549	474	418	373	337	307	282	261	243	227	213	201	190	180	171
10-#9	.0174	.164	589	506	444	395	356	324	297	274	255	238	223	210	199	188	179
11-#9	.0191	.166	609	522	457	406	366	333	305	282	262	245	229	215	204	193	183
12-#9	.0208	.169	629	538	471	418	376	341	313	288	268	250	234	220	208	197	187
14-#9	.0243	.176	669	569	496	438	393	356	326	300	278	259	242	228	215	204	193
6-#10	.0132	.155	541	468	413	369	334	305	280	259	241	226	212	200	189	179	171
7-#10	.0154	.160	567	489	430	384	346	315	289	268	249	232	218	206	194	184	175
8-#10	.0176	.164	592	508	446	397	358	325	298	276	256	239	224	211	199	189	179
9-#10	.0198	.167	618	531	464	412	371	337	309	285	265	247	232	218	206	195	186
10-#10	.0220	.171	643	551	481	426	382	342	318	293	272	254	238	224	211	200	190
12-#10	.0265	.180	694	588	511	452	404	365	334	307	284	265	248	233	220	208	197
13-#10	.0286	.182	719	608	528	466	417	377	344	316	293	273	255	240	226	214	203
6-#11	.0162	.161	576	496	436	388	351	319	293	271	252	235	221	208	197	186	177
7-#11	.0190	.166	607	522	457	406	365	332	305	281	261	244	229	215	203	193	183
8-#11	.0217	.171	639	547	476	423	380	345	316	291	270	252	236	222	210	198	189
9-#11	.0244	.176	670	569	496	438	393	357	326	300	279	259	243	229	216	204	193
10-#11	.0271	.180	701	594	516	456	408	369	337	310	287	268	251	235	222	210	199
11-#11	.0298	.186	732	618	534	470	420	379	346	318	294	274	256	240	227	215	203
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—25" x 25"

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#7	.0115	.144	566	495	439	395	359	329	303	282	263	246	232	219	207	197	188
8-#8	.0101	.142	548	480	427	384	349	320	296	275	257	241	226	214	203	193	183
10-#8	.0126	.147	580	506	448	402	365	334	308	286	266	249	235	222	210	199	189
12-#8	.0151	.154	612	530	468	418	378	345	318	294	274	256	241	227	215	203	194
6-#9	.0096	.142	542	474	422	380	345	317	292	272	254	238	224	211	200	190	181
8-#9	.0128	.149	582	507	448	402	365	334	307	285	266	248	234	221	209	198	189
10-#9	.0160	.155	622	538	477	425	384	350	322	298	278	260	244	230	217	206	196
11-#9	.0176	.157	642	555	489	436	395	359	330	306	285	266	250	235	222	211	201
12-#9	.0192	.160	662	570	502	447	403	368	338	312	291	271	255	240	227	215	204
14-#9	.0224	.165	702	602	527	470	422	385	353	326	303	282	265	249	236	223	212
6-#10	.0122	.145	574	501	445	400	363	333	307	285	266	249	234	221	209	198	189
7-#10	.0142	.152	600	520	460	412	373	341	314	290	271	253	238	224	212	202	192
8-#10	.0162	.155	625	541	477	427	386	352	324	300	279	261	245	231	219	207	197
9-#10	.0184	.159	651	561	494	440	398	362	333	308	287	268	252	237	224	212	200
10-#10	.0203	.161	676	582	511	456	411	374	343	318	295	276	259	244	230	218	207
11-#10	.0223	.165	701	602	527	469	422	384	352	325	302	282	264	249	235	223	212
12-#10	.0244	.169	727	622	543	483	434	394	361	333	309	288	270	254	240	228	216
13-#10	.0264	.172	752	642	559	495	445	404	370	341	316	295	276	260	245	232	220
6-#11	.0150	.154	609	528	466	416	377	344	316	293	272	255	240	226	214	203	193
7-#11	.0174	.157	640	554	487	435	393	358	329	305	283	265	249	235	222	210	200
8-#11	.0200	.161	672	579	509	453	409	372	342	316	294	274	258	243	229	217	206
9-#11	.0225	.165	703	603	528	470	423	385	353	326	303	283	265	250	236	224	212
10-#11	.0250	.170	734	627	548	486	437	397	363	335	311	290	272	255	241	228	217
11-#11	.0274	.173	765	652	568	504	452	410	375	346	321	299	280	263	249	235	223
12-#11	.0300	.179	796	675	586	518	463	420	383	353	327	304	285	268	253	239	227
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—26″ x 26″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#8	.0105	.138	598	527	469	424	386	354	328	304	284	267	252	238	225	214	204
11-#8	.0128	.142	630	551	491	443	403	368	340	316	295	277	261	246	233	221	211
13-#8	.0152	.148	661	576	511	457	415	380	350	325	303	284	267	252	238	226	215
15-#8	.0175	.151	693	602	532	477	432	395	364	338	314	294	275	260	247	234	223
7-#9	.0103	.137	596	525	468	423	386	354	328	305	285	267	252	238	226	214	205
8-#9	.0118	.139	616	542	483	435	396	364	336	313	292	274	258	244	231	220	209
9-#9	.0133	.144	636	556	495	444	404	370	341	317	296	278	261	246	233	222	211
10-#9	.0148	.148	656	572	507	455	412	377	348	323	300	282	265	250	236	224	214
11-#9	.0163	.149	676	589	521	468	424	388	357	331	309	288	272	256	242	230	219
12-#9	.0177	.151	696	605	535	480	434	397	366	328	316	296	278	268	248	235	224
13-#9	.0192	.153	716	620	549	491	444	406	374	346	322	303	283	267	253	240	228
15-#9	.0222	.158	756	653	575	513	463	422	388	359	334	312	293	276	261	248	236
6-#10	.0113	.138	608	535	477	430	392	360	333	310	289	271	256	242	229	218	208
8-#10	.0150	.148	659	575	509	457	414	379	350	324	302	283	266	251	238	225	215
10-#10	.0188	.153	710	615	544	487	440	402	371	343	319	299	281	264	251	238	226
12-#10	.0226	.159	761	657	578	515	465	424	390	360	335	313	294	277	262	248	236
14-#10	.0263	.165	812	696	610	543	489	445	408	377	350	327	306	289	272	258	245
6-#11	.0138	.145	643	562	498	448	407	373	344	320	298	279	263	248	235	223	212
7-#11	.0161	.149	674	586	519	466	422	386	356	329	307	288	271	256	242	230	218
8-#11	.0184	.152	706	613	541	485	440	402	370	342	319	298	280	264	250	237	226
9-#11	.0208	.156	737	638	561	503	454	415	381	353	328	306	288	272	257	244	232
10-#11	.0230	.159	768	663	583	520	470	428	393	364	338	316	296	280	264	250	238
11-#11	.0254	.164	799	686	602	535	483	439	403	372	346	323	303	284	269	255	242
13-#11	.0300	.171	862	735	642	570	512	465	425	393	364	340	318	299	283	267	254
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—27″ x 27″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
14-#8	.0153	.143	713	624	552	499	453	416	383	356	332	312	293	277	262	249	237
16-#8	.0174	.145	745	651	578	519	472	432	398	370	345	323	304	287	272	258	246
18-#8	.0196	.148	776	675	598	537	487	446	411	381	355	332	312	295	279	265	252
10-#9	.0137	.140	692	607	540	487	443	407	376	349	326	306	288	272	258	245	233
12-#9	.0165	.144	732	640	568	511	464	425	392	364	340	319	300	283	268	255	242
14-#9	.0193	.147	772	673	598	538	486	445	410	380	354	332	312	295	279	265	252
16-#9	.0220	.152	812	705	622	558	505	461	424	393	366	343	322	303	287	273	259
6-#10	.0150	.132	644	569	509	461	421	388	359	334	313	294	277	262	249	237	226
7-#10	.0122	.134	670	590	528	478	436	401	371	345	323	304	286	271	257	244	233
8-#10	.0139	.140	695	610	543	489	445	408	377	351	328	307	289	273	259	246	235
9-#10	.0158	.143	721	630	560	504	458	420	388	360	336	315	297	280	266	252	240
10-#10	.0174	.145	746	651	578	520	472	432	399	370	345	323	304	287	272	258	246
11-#10	.0192	.147	771	672	596	535	485	444	409	380	354	331	312	295	279	265	252
12-#10	.0209	.150	797	692	612	550	498	455	420	389	362	339	319	301	284	270	257
13-#10	.0227	.153	822	713	629	563	510	465	428	396	369	346	325	306	290	275	261
14-#10	.0243	.156	848	733	646	577	521	476	438	405	377	352	331	312	295	280	266
15-#10	.0262	.158	873	754	663	592	535	487	445	414	385	360	338	319	301	286	272
6-#11	.0128	.138	679	596	532	480	437	401	371	345	322	302	285	269	255	243	231
8-#11	.0172	.145	742	648	575	517	469	430	396	368	343	322	302	286	271	257	245
9-#11	.0193	.147	773	674	597	536	486	445	410	381	355	332	313	295	279	265	253
10-#11	.0214	.149	804	700	619	555	504	460	424	393	367	343	322	304	288	274	260
11-#11	.0236	.154	835	723	638	571	516	471	433	401	374	350	328	310	293	278	264
12-#11	.0257	.158	866	747	658	587	530	483	444	411	382	357	335	316	299	283	269
13-#11	.0277	.161	898	774	679	605	546	497	456	422	392	367	344	324	306	290	276
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—28" x 28"

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#8	.0101	.127	687	610	547	497	456	420	390	364	341	320	302	287	272	259	248
12-#8	.0121	.129	719	638	572	519	475	438	405	378	354	333	314	298	282	269	256
14-#8	.0141	.135	750	660	590	534	487	448	415	386	360	339	319	302	286	272	260
8-#9	.0102	.127	689	612	550	499	458	422	391	366	332	322	304	288	273	260	248
10-#9	.0128	.132	729	644	576	521	477	439	406	379	355	333	314	297	282	268	256
12-#9	.0153	.137	769	676	604	545	497	456	421	392	367	346	324	306	291	276	264
14-#9	.0179	.141	809	709	631	568	517	475	439	407	380	357	336	317	301	286	272
7-#10	.0113	.128	707	628	564	511	467	431	400	373	349	329	310	294	279	265	254
8-#10	.0130	.133	732	646	579	524	478	440	408	379	355	334	314	297	282	268	256
9-#10	.0146	.136	758	668	595	539	490	451	418	388	363	340	321	304	288	274	261
10-#10	.0162	.138	783	690	615	554	505	464	429	399	372	349	329	312	295	281	267
11-#10	.0178	.141	808	708	630	568	517	475	438	407	380	356	336	317	301	286	272
12-#10	.0194	.142	834	730	650	585	532	488	450	418	391	367	345	325	308	293	279
13-#10	.0211	.146	859	750	665	597	542	497	458	424	396	372	349	330	312	296	282
14-#10	.0227	.148	885	771	683	613	555	509	470	435	405	379	357	337	319	302	288
6-#11	.0120	.129	716	635	570	517	472	435	403	376	353	331	313	296	281	268	255
7-#11	.0139	.135	747	658	588	531	485	446	412	384	359	337	318	300	285	271	258
8-#11	.0159	.138	779	685	610	551	502	461	427	396	370	347	327	310	294	279	266
9-#11	.0179	.141	810	710	631	569	518	475	439	408	380	357	336	318	301	286	273
10-#11	.0199	.143	841	735	654	590	535	490	453	420	392	368	346	327	310	294	280
11-#11	.0219	.147	872	760	674	604	550	503	463	430	401	374	353	334	315	300	285
12-#11	.0239	.150	903	786	695	624	565	516	476	442	411	385	362	341	323	306	292
13-#11	.0259	.153	935	811	716	642	580	530	488	452	420	394	370	348	330	313	298
14-#11	.0318	.163	966	830	730	650	585	532	489	451	419	392	368	346	327	310	294
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—29″ x 29″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
19-#8	.0178	.133	868	766	675	620	566	521	483	449	420	395	373	352	334	318	303
21-#8	.0198	.136	899	791	706	638	582	535	495	460	430	404	381	360	341	325	309
11-#9	.0131	.126	787	698	628	571	523	483	448	418	392	369	348	330	313	298	285
12-#9	.0143	.129	807	715	642	581	532	491	455	424	397	373	352	334	317	302	288
13-#9	.0154	.131	827	731	655	594	543	499	463	431	404	380	358	338	321	306	292
14-#9	.0166	.132	847	749	670	607	554	510	473	440	412	387	365	345	328	312	298
15-#9	.0178	.134	867	765	684	618	564	519	480	447	418	393	370	350	333	316	303
16-#9	.0190	.135	887	781	698	631	575	529	490	455	426	401	377	357	338	322	307
17-#9	.0202	.136	907	798	713	644	588	540	499	465	434	408	384	363	344	328	312
18-#9	.0215	.139	927	814	725	654	595	547	506	470	438	411	388	367	347	330	314
8-#10	.0120	.124	770	685	617	561	514	475	442	412	386	364	344	326	310	295	281
9-#10	.0136	.127	795	705	634	575	527	486	451	421	394	371	350	332	315	300	286
10-#10	.0151	.130	821	726	652	591	539	497	460	429	402	378	356	337	320	304	291
11-#10	.0166	.132	846	747	670	606	554	510	472	440	412	387	365	345	327	312	297
12-#10	.0184	.134	872	869	688	617	567	522	483	450	421	395	373	352	334	318	303
13-#10	.0196	.136	897	889	705	637	581	534	494	459	430	403	380	359	341	324	309
14-#10	.0210	.137	922	811	724	653	595	547	506	471	440	413	389	368	348	331	316
15-#10	.0232	.140	948	831	740	668	608	558	515	479	447	419	395	373	354	336	320
16-#10	.0242	.142	973	852	758	682	621	569	525	488	456	427	402	379	360	342	326
8-#11	.0148	.130	817	722	648	588	536	494	458	427	399	375	354	335	318	303	289
9-#11	.0167	.132	848	749	671	607	555	511	473	441	413	388	365	346	328	312	298
10-#11	.0186	.135	879	774	692	626	571	525	486	452	423	397	374	354	335	319	304
11-#11	.0205	.137	911	801	715	645	588	541	500	465	435	408	384	363	344	327	312
12-#11	.0223	.140	942	826	736	663	604	554	512	476	444	417	392	371	351	334	318
13-#11	.0240	.142	973	852	758	682	621	569	525	488	456	427	402	379	360	342	326
14-#11	.0260	.145	1004	877	779	700	636	582	537	498	465	436	410	387	366	348	331
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

f'_c = 3000 psi f_s = 20,000 psi

COLUMN SIZE—30″ x 30″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#9	.0100	.117	788	705	639	583	538	497	463	434	407	384	363	345	327	312	299
11-#9	.0122	.120	828	740	668	610	560	517	481	450	422	398	376	356	340	324	309
13-#9	.0144	.124	868	773	696	633	580	536	498	465	436	411	388	367	349	333	317
15-#9	.0167	.128	908	805	723	655	600	554	514	479	448	422	398	377	358	340	328
8-#10	.0113	.119	811	725	655	598	550	509	474	443	415	392	370	352	334	318	304
9-#10	.0127	.121	837	746	674	614	564	521	485	454	425	400	379	359	341	326	311
10-#10	.0141	.124	862	766	690	628	576	531	494	461	432	408	384	364	346	330	315
11-#10	.0155	.126	887	788	709	645	590	550	505	471	442	416	393	372	354	337	321
12-#10	.0169	.128	913	810	729	660	604	557	517	482	451	425	400	379	360	343	327
13-#10	.0183	.130	938	830	745	675	617	568	527	491	460	432	408	386	366	349	333
14-#10	.0198	.132	964	850	763	690	631	580	538	500	469	440	415	393	372	355	338
15-#10	.0212	.133	989	871	781	707	645	594	550	511	478	450	424	401	381	362	345
16-#10	.0226	.136	1014	893	796	721	656	604	559	520	486	456	430	407	386	366	349
17-#10	.0240	.137	1040	915	816	736	672	618	570	531	496	466	439	415	393	374	357
6-#11	.0104	.117	795	712	644	588	542	501	467	437	411	388	366	348	330	315	302
7-#11	.0121	.120	826	738	666	608	558	516	480	449	422	397	376	356	338	323	308
8-#11	.0139	.124	858	764	688	625	574	530	492	460	430	406	383	362	345	329	314
9-#11	.0156	.126	889	790	710	645	590	545	506	472	443	417	393	372	354	337	321
10-#11	.0173	.128	920	816	733	665	609	561	521	486	455	428	404	382	363	345	330
11-#11	.0190	.131	951	841	754	683	624	575	533	496	464	437	412	389	370	352	336
12-#11	.0208	.133	982	866	775	702	640	590	546	509	475	447	422	398	378	360	343
13-#11	.0225	.136	1014	894	798	721	657	604	559	520	486	455	430	407	386	366	349
14-#11	.0242	.137	1045	920	820	740	676	620	574	534	499	469	441	417	395	375	358
15-#11	.0260	.140	1076	945	841	758	690	634	585	544	508	476	449	424	402	382	364
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—31″ x 31″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
16-#8	.0132	.118	901	805	729	665	612	566	527	493	463	436	413	392	373	355	339
20-#8	.0165	.123	964	858	773	704	646	596	554	518	486	458	432	409	389	371	354
12-#9	.0125	.117	888	795	720	657	605	560	522	488	458	432	409	388	369	352	337
14-#9	.0146	.120	928	828	748	682	627	580	539	504	473	446	422	400	380	362	346
16-#9	.0167	.123	968	862	777	707	647	600	557	520	488	460	434	412	391	373	355
18-#9	.0188	.126	1008	895	805	732	671	619	574	536	502	473	446	423	401	382	365
20-#9	.0208	.129	1048	929	834	756	691	637	591	551	515	485	458	439	412	392	374
8-#10	.0105	.113	851	765	694	636	586	544	507	475	447	422	399	379	361	345	329
10-#10	.0132	.118	902	807	729	666	613	567	528	494	464	437	414	393	373	356	340
11-#10	.0145	.120	927	828	747	682	626	579	539	504	473	445	421	399	380	362	346
12-#10	.0159	.123	953	849	765	696	639	590	548	512	480	452	427	410	385	367	350
13-#10	.0172	.124	978	870	783	713	654	604	551	524	491	462	437	413	393	374	357
14-#10	.0185	.126	1003	891	802	728	668	616	572	536	501	470	444	421	399	380	363
15-#10	.0199	.128	1029	912	817	743	681	627	582	542	508	477	451	427	412	386	368
16-#10	.0212	.129	1054	934	838	760	695	641	594	554	518	488	460	436	414	394	374
17-#10	.0225	.130	1080	956	857	778	711	655	607	566	530	498	469	445	422	403	383
18-#10	.0238	.133	1105	976	873	791	722	664	615	572	536	503	474	449	426	410	386
8-#11	.0130	.118	898	803	726	663	610	564	525	491	462	435	412	391	372	354	338
9-#11	.0146	.121	929	829	748	682	626	579	538	503	472	445	420	398	379	361	345
10-#11	.0162	.123	960	855	770	701	643	594	552	515	484	456	430	407	388	369	352
11-#11	.0178	.125	992	882	793	721	661	671	567	529	495	467	440	417	397	378	360
12-#11	.0195	.127	1023	907	816	741	679	626	581	542	508	477	451	427	410	386	368
13-#11	.0210	.128	1054	934	839	761	697	643	596	556	520	489	462	438	416	395	377
14-#11	.0228	.131	1085	959	860	779	712	655	607	566	530	498	470	444	422	401	383
15-#11	.0244	.134	1116	985	880	796	726	668	619	576	538	506	477	451	428	414	388
16-#11	.0260	.136	1148	1011	903	816	744	684	632	588	550	516	487	460	436	415	395
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—32″ x 32″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#9	.0117	.113	931	836	760	695	640	595	555	520	489	462	437	415	396	377	360
14-#9	.0137	.116	971	870	787	720	664	614	573	535	504	475	450	427	406	387	370
16-#9	.0156	.118	1011	906	820	747	687	636	594	555	520	490	465	441	420	400	382
18-#9	.0176	.121	1051	938	846	771	709	655	610	570	535	504	475	451	429	409	391
20-#9	.0195	.124	1091	970	875	795	730	674	626	585	548	516	487	461	439	418	400
10-#10	.0124	.113	945	849	771	706	650	604	564	527	496	469	444	421	402	383	366
11-#10	.0136	.115	970	871	789	721	665	616	575	537	505	477	451	428	408	389	372
12-#10	.0149	.117	996	893	807	736	680	628	585	548	515	485	459	435	414	395	378
13-#10	.0161	.119	1021	914	825	754	692	640	596	557	523	494	466	443	421	401	383
14-#10	.0173	.120	1047	935	845	770	707	654	608	569	534	503	476	451	429	408	390
15-#10	.0186	.123	1072	955	860	785	719	664	617	575	541	509	481	456	434	414	394
16-#10	.0198	.124	1097	975	880	799	734	676	629	588	550	519	490	464	441	420	402
17-#10	.0211	.125	1123	1002	900	818	750	692	642	600	562	529	500	474	450	428	409
18-#10	.0224	.126	1148	1020	916	835	764	705	655	610	571	539	508	481	458	436	415
8-#11	.0122	.113	941	845	766	704	648	601	561	525	494	467	441	419	400	381	364
9-#11	.0137	.115	972	871	790	722	665	616	575	538	506	478	452	428	408	390	372
10-#11	.0152	.117	1003	900	814	743	685	634	590	553	519	490	463	438	418	398	381
11-#11	.0167	.120	1034	924	835	760	700	647	601	567	528	497	470	446	424	404	386
12-#11	.0183	.122	1065	950	856	780	716	662	615	575	540	509	480	455	432	412	394
13-#11	.0198	.124	1097	975	880	800	734	678	629	588	550	520	490	464	441	420	401
14-#11	.0213	.125	1128	1003	901	820	750	694	645	601	564	530	501	475	451	430	410
15-#11	.0228	.127	1159	1030	924	839	769	709	657	614	575	540	510	484	459	437	418
16-#11	.0244	.129	1190	1054	946	859	785	724	671	625	586	550	520	493	467	445	424
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—33″ x 33″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
13-#9	.0119	.109	995	900	819	751	694	645	602	565	531	503	476	454	431	413	394
15-#9	.0138	.112	1035	930	845	775	715	664	619	580	546	516	488	464	441	422	403
17-#9	.0156	.115	1075	965	875	800	737	683	636	596	560	529	501	475	452	432	412
19-#9	.0175	.117	1115	998	904	825	760	704	655	614	576	544	514	487	464	443	423
10-#10	.0117	.108	989	894	814	746	690	642	601	564	530	501	475	453	431	411	394
11-#10	.0128	.110	1014	915	831	763	705	655	611	573	540	510	484	459	438	418	400
12-#10	.0140	.113	1040	934	849	778	715	665	620	580	546	516	488	463	442	421	403
13-#10	.0152	.114	1065	956	869	794	732	679	633	593	557	525	498	474	450	429	411
14-#10	.0163	.115	1091	980	888	811	748	694	646	605	569	537	508	482	459	438	419
15-#10	.0175	.117	1116	1000	905	825	761	705	655	615	576	545	515	488	464	443	424
16-#10	.0187	.118	1141	1021	925	844	775	719	670	626	587	554	524	498	473	451	431
17-#10	.0198	.120	1167	1041	940	856	789	730	678	634	595	561	530	503	478	455	435
18-#10	.0210	.121	1192	1061	960	875	804	743	691	646	606	571	540	511	487	464	443
19-#10	.0221	.123	1218	1083	978	890	816	755	701	654	614	578	546	518	492	470	447
8-#11	.0115	.108	985	890	810	744	688	640	599	562	529	500	474	450	430	409	393
9-#11	.0129	.110	1016	915	834	764	706	655	613	574	540	510	484	460	438	418	400
10-#11	.0143	.113	1047	940	854	782	720	669	625	584	550	520	491	466	444	424	405
11-#11	.0157	.115	1078	967	877	801	740	685	638	597	561	530	502	475	453	432	414
12-#11	.0172	.116	1109	994	900	824	757	702	654	611	575	542	514	488	464	442	422
13-#11	.0186	.118	1141	1021	924	842	725	718	669	625	587	553	524	497	475	451	430
14-#11	.0201	.120	1172	1048	946	863	793	734	682	637	598	564	534	505	480	458	438
15-#11	.0215	.122	1203	1072	968	881	810	748	695	650	610	574	542	514	488	465	445
16-#11	.0229	.124	1234	1098	990	900	825	762	708	661	620	584	551	522	496	474	452
17-#11	.0244	.125	1266	1126	1012	920	844	780	724	675	633	595	563	534	506	482	460
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—14″ x 14″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#5	.0095	.254	202	161	134	114	100	89	80	72	66	61	29	24	21	18	16
8-#5	.0126	.262	215	171	141	120	105	93	84	76	71	64	36	31	27	23	21
10-#5	.0158	.273	227	178	147	125	109	96	86	78	71	66	41	35	31	27	25
6-#6	.0135	.266	218	172	142	122	106	94	84	76	70	64	38	31	28	25	22
7-#6	.0157	.273	227	179	147	125	109	96	86	78	72	66	40	35	31	27	25
8-#6	.0180	.283	235	183	150	127	110	97	87	79	72	66	42	38	34	30	27
9-#6	.0202	.291	244	189	154	130	113	99	89	80	73	67	44	40	36	33	30
6-#7	.0184	.284	237	185	151	128	111	98	88	79	73	67	42	38	35	31	27
7-#7	.0214	.296	249	193	157	132	114	101	90	81	74	68	45	40	37	34	31
8-#7	.0254	.310	261	199	161	135	117	102	91	82	75	69	47	43	39	36	33
6-#8	.0252	.309	260	199	161	135	117	102	91	82	75	69	47	43	39	36	33
7-#8	.0282	.319	276	210	169	141	122	107	95	86	78	71	50	45	41	38	35
8-#8	.0322	.331	291	219	175	146	125	110	98	88	80	73	53	48	44	40	37
6-#9	.0306	.326	285	215	173	144	124	109	97	87	79	73	52	47	43	39	36
7-#9	.0357	.341	305	228	182	151	129	113	100	93	82	75	55	50	46	42	39
6-#10	.0390	.350	317	235	187	155	132	115	103	92	84	77	57	52	48	44	41
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	†	†	†	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—15″ x 15″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0117	.239	243	196	164	142	124	111	100	91	83	77	72	38	32	28	25
9-#6	.0176	.263	269	213	176	150	131	116	104	95	87	80	74	48	44	39	35
6-#7	.0160	.258	262	208	173	148	129	114	103	93	86	79	73	47	39	36	32
7-#7	.0187	.267	274	216	178	152	132	117	105	95	87	80	75	49	45	41	36
8-#7	.0213	.275	286	224	184	157	136	120	108	98	89	82	76	52	47	43	40
9-#7	.0240	.284	298	232	190	161	140	123	110	100	91	84	78	54	49	45	42
6-#8	.0211	.275	285	223	184	156	136	120	107	97	89	82	76	52	47	43	40
7-#8	.0246	.286	301	234	191	162	140	124	111	100	91	84	78	55	50	46	42
8-#8	.0281	.297	316	244	198	167	144	127	114	103	94	86	80	58	53	48	45
6-#9	.0267	.292	310	240	195	165	143	126	113	102	93	85	79	56	51	47	44
7-#9	.0311	.306	330	253	205	172	148	130	116	105	96	88	81	60	55	50	47
8-#9	.0355	.319	350	265	214	179	154	135	120	108	98	90	84	64	58	54	50
6-#10	.3339	.314	342	260	210	176	152	133	118	107	97	89	83	63	57	53	49
6-#11	.0416	.336	377	282	225	188	161	141	125	112	102	94	86	68	63	58	53
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	†	†	†	†

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—16″ x 16″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0103	.223	269	220	186	161	143	127	115	105	98	90	84	45	41	33	29
9-#6	.0155	.239	295	238	200	172	151	134	122	111	102	94	87	57	51	45	39
6-#7	.0140	.234	288	234	196	170	149	133	120	109	101	93	87	54	45	39	35
7-#7	.0164	.241	300	240	203	175	153	136	123	112	103	95	88	58	53	46	41
9-#7	.0210	.254	324	259	215	184	161	143	129	117	107	99	92	65	59	54	50
11-#7	.0258	.268	348	274	227	193	168	149	133	121	111	103	95	70	64	59	54
6-#8	.0185	.247	311	249	209	179	157	139	126	114	105	97	90	61	56	51	45
7-#8	.0216	.255	327	261	217	185	162	144	130	118	108	99	92	65	59	54	50
8-#8	.0246	.265	342	270	224	190	166	147	132	120	110	101	94	68	62	57	53
9-#8	.0278	.273	358	282	231	197	171	151	136	123	113	103	96	72	66	60	56
6-#9	.0234	.262	336	267	221	188	164	146	131	119	109	100	93	67	61	56	52
7-#9	.0273	.272	356	280	231	196	171	151	136	123	112	103	96	71	65	60	55
8-#9	.0312	.281	376	294	241	204	177	156	140	127	116	107	99	75	69	63	59
6-#10	.0298	.279	368	287	236	200	173	154	138	125	115	105	97	74	67	62	57
7-#10	.0347	.289	394	307	250	211	182	161	144	130	119	109	101	79	72	66	62
6-#11	.0366	.293	403	313	254	215	186	164	147	133	121	111	103	80	73	68	63
7-#11	.0426	.309	434	332	268	227	194	171	152	137	125	115	106	87	79	73	67
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	†	†	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—17″ x 17″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0125	.215	316	260	221	192	170	152	138	126	116	108	100	94	55	47	42
9-#7	.0187	.233	352	285	240	207	182	163	147	134	123	114	106	99	69	63	58
11-#7	.0228	.244	376	302	253	217	190	169	152	139	127	118	109	102	75	68	63
6-#8	.0164	.226	339	277	234	202	178	159	144	131	121	112	104	97	65	58	51
8-#8	.0219	.241	370	298	250	215	188	168	151	138	126	117	108	101	74	67	62
10-#8	.0273	.255	402	320	266	228	199	176	159	144	132	122	113	106	80	73	68
6-#9	.0208	.238	364	294	247	212	186	166	150	136	125	116	108	101	72	66	61
8-#9	.0277	.256	404	322	267	228	199	177	159	145	132	122	113	106	80	73	68
10-#9	.0346	.273	444	348	287	244	212	188	168	153	139	128	119	111	88	81	75
6-#10	.0264	.253	396	316	263	225	197	175	157	143	131	121	112	105	78	72	66
7-#10	.0308	.264	422	334	276	235	205	182	163	148	136	125	116	108	85	78	72
8-#10	.0352	.274	447	350	288	245	213	188	169	153	140	129	119	111	89	82	76
6-#11	.0324	.268	431	340	281	239	208	184	165	150	137	126	117	109	86	79	73
7-#11	.0378	.280	462	361	296	251	218	192	172	156	142	131	121	113	92	85	78
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.975	.975	.980	.985	†	†	†

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—18″ x 18″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0111	.200	345	289	246	216	192	173	157	144	133	123	115	108	102	56	50
9-#7	.0167	.215	381	314	267	232	205	184	167	152	140	130	121	113	107	74	66
11-#7	.0204	.225	405	331	280	242	213	191	172	157	145	134	125	117	110	79	73
6-#8	.0146	.209	368	304	259	226	200	180	164	150	138	128	119	111	105	67	60
7-#8	.0171	.215	384	316	269	233	206	185	168	153	141	131	122	114	107	74	66
8-#8	.0195	.222	399	327	277	239	211	189	172	157	144	133	124	116	109	78	72
9-#8	.0220	.228	415	339	285	247	217	194	175	160	147	136	127	118	111	81	75
11-#8	.0268	.241	447	360	302	260	228	203	183	167	153	141	131	124	115	88	82
6-#9	.0185	.222	393	322	272	236	208	186	169	154	142	131	122	114	108	77	71
7-#9	.0216	.227	413	337	285	246	217	194	175	160	147	136	126	118	111	81	75
8-#9	.0247	.238	433	350	294	254	222	198	179	163	149	138	128	120	113	86	79
9-#9	.0278	.244	453	365	305	262	230	205	184	167	154	142	132	123	115	90	83
10-#9	.0308	.250	473	379	316	272	237	210	190	172	158	146	135	127	119	94	87
6-#10	.0235	.235	425	344	289	249	219	196	179	161	148	137	127	119	111	84	77
7-#10	.0274	.242	451	364	304	261	229	204	184	168	154	142	132	123	116	89	82
8-#10	.0314	.251	476	380	317	272	238	211	190	173	159	146	136	127	119	94	87
9-#10	.0353	.258	502	399	331	283	248	220	197	179	164	152	140	131	123	100	92
6-#11	.0289	.247	460	368	309	264	231	206	186	169	155	143	133	124	116	91	84
7-#11	.0338	.256	491	391	325	278	243	215	194	176	161	149	138	129	120	98	91
8-#11	.0385	.265	523	414	342	292	254	226	202	184	168	155	144	134	125	104	96
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\quad$ $f_s = 20{,}000$ psi

COLUMN SIZE—19″ x 19″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0131	.192	400	335	289	254	226	204	186	170	158	146	137	129	121	81	67
7-#8	.0153	.198	416	348	298	261	232	209	190	174	161	150	140	131	123	86	76
8-#8	.0175	.203	431	358	307	268	238	214	194	178	164	152	142	133	126	90	84
9-#8	.0197	.208	447	370	316	275	244	219	199	182	168	156	145	136	128	94	87
10-#8	.0219	.213	463	381	325	282	250	224	203	186	171	159	148	138	130	98	91
11-#8	.0241	.218	479	393	334	289	256	229	207	190	175	162	151	141	132	102	94
6-#9	.0166	.200	425	354	303	265	236	212	192	177	163	152	142	133	125	89	80
7-#9	.0194	.207	445	368	314	274	243	219	198	182	168	155	145	136	128	94	87
8-#9	.0221	.213	465	383	326	284	241	225	204	186	172	159	148	139	131	99	91
9-#9	.0249	.219	485	398	337	292	258	231	210	192	176	163	152	142	134	103	96
10-#9	.0277	.225	505	412	348	301	268	237	215	196	180	167	155	145	136	108	101
11-#9	.0305	.231	525	426	359	310	273	243	220	200	184	171	158	148	139	113	104
6-#10	.0211	.211	457	377	321	280	248	222	202	184	170	158	147	137	129	97	90
7-#10	.0246	.219	483	396	336	291	257	230	209	191	176	162	151	142	133	103	95
8-#10	.0281	.226	508	414	350	303	267	238	216	197	181	167	156	146	137	108	100
9-#10	.0316	.233	534	432	364	314	276	246	223	203	186	172	160	150	141	115	106
10-#10	.0352	.240	559	450	378	325	285	254	229	208	191	177	164	154	144	120	111
6-#11	.0259	.222	492	402	341	295	261	233	211	193	177	164	153	143	134	105	97
7-#11	.0302	.231	523	425	358	308	272	242	219	200	184	170	158	148	139	112	103
8-#11	.0346	.239	555	448	375	323	284	253	228	208	191	176	164	153	143	119	110
9-#11	.0388	.246	586	470	393	337	295	263	237	215	197	182	169	158	148	127	117
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	†	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—20″ x 20″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0118	.179	432	366	319	282	252	229	208	191	178	165	155	145	137	130	72
7-#8	.0138	.184	448	379	328	289	259	234	214	196	182	169	158	149	140	133	84
8-#8	.0158	.189	463	390	336	296	264	239	218	200	185	172	161	151	142	134	95
9-#8	.0178	.192	479	402	346	304	271	244	223	204	189	176	164	154	145	137	99
10-#8	.0198	.198	495	412	354	310	276	249	226	207	192	178	166	156	147	138	103
11-#8	.0217	.201	511	426	365	319	284	255	232	212	196	182	170	159	150	142	107
12-#8	.0237	.206	527	437	374	326	290	259	236	216	199	185	172	161	152	144	111
6-#9	.0150	.186	457	384	334	293	262	237	217	199	183	172	160	152	142	134	91
7-#9	.0175	.192	477	400	345	303	271	243	222	204	188	175	164	153	145	137	98
8-#9	.0200	.198	497	415	355	311	277	250	228	208	192	179	167	157	147	139	104
9-#9	.0225	.203	517	430	368	321	286	255	233	214	197	183	170	160	150	142	109
10-#9	.0250	.209	537	443	379	329	292	263	237	218	201	186	174	163	152	144	114
11-#9	.0275	.215	557	458	389	338	299	277	242	222	204	190	177	166	155	147	119
6-#10	.0191	.195	489	410	351	308	274	247	226	207	191	177	165	155	147	139	101
7-#10	.0222	.202	515	429	367	320	286	257	234	214	198	183	171	161	151	142	108
8-#10	.0254	.212	540	446	380	331	292	263	238	218	200	186	173	162	153	144	115
9-#10	.0286	.218	566	464	394	343	303	271	245	224	206	191	178	167	157	148	121
10-#10	.0317	.222	591	484	409	355	313	281	254	232	213	197	184	172	162	152	127
6-#11	.0234	.206	524	434	370	324	288	258	235	215	198	184	172	161	151	143	110
7-#11	.0273	.214	555	457	389	338	299	268	244	222	205	190	177	166	156	147	118
8-#11	.0312	.221	587	481	407	353	312	279	252	230	212	196	183	171	161	151	126
9-#11	.0351	.228	618	504	426	367	323	289	261	239	219	203	189	177	166	156	134
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	†

Outside diameter of spiral should be 3 in. less than side of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—21″ x 21″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0108	.169	467	399	349	310	279	253	232	214	199	185	174	163	154	146	139
7-#8	.0125	.173	483	411	359	318	285	259	237	218	202	189	177	166	157	148	141
8-#8	.0143	.176	498	424	368	326	292	265	242	223	207	193	180	170	160	152	144
9-#8	.0161	.181	514	435	377	333	298	270	246	227	210	195	183	172	162	153	145
10-#8	.0179	.184	530	447	387	341	305	276	252	232	214	200	186	175	165	156	148
11-#8	.0197	.187	546	460	397	350	312	282	257	236	219	203	190	179	168	159	151
12-#8	.0215	.191	562	472	407	357	318	287	262	241	222	207	193	181	171	161	153
6-#9	.0136	.175	492	419	364	322	289	262	240	221	205	191	179	168	159	150	143
7-#9	.0159	.180	512	434	376	332	298	270	246	226	210	195	183	172	162	153	145
8-#9	.0181	.185	532	449	388	342	306	276	252	232	215	200	187	175	165	156	148
9-#9	.0204	.189	552	464	400	352	314	284	259	238	220	204	191	179	169	160	152
10-#9	.0227	.194	572	479	412	362	322	290	264	243	224	208	195	183	172	162	154
11-#9	.0249	.198	592	494	424	371	330	298	270	248	229	213	199	186	175	166	157
12-#9	.0272	.203	612	509	436	380	338	304	276	253	233	216	202	189	178	168	159
6-#10	.0173	.183	524	442	384	338	302	274	250	230	213	198	185	174	164	155	147
7-#10	.0202	.189	550	462	399	351	313	283	258	236	219	204	190	179	168	159	151
8-#10	.0230	.195	575	481	413	363	323	291	265	243	225	209	195	183	172	162	154
9-#10	.0259	.200	601	501	429	376	334	300	273	250	231	215	200	188	177	167	158
10-#10	.0288	.206	626	519	443	387	343	308	280	256	236	219	205	192	180	170	161
6-#11	.0212	.191	559	469	404	355	316	286	260	239	221	206	192	180	170	160	152
7-#11	.0248	.198	590	492	423	370	329	296	269	247	228	212	198	186	175	165	156
8-#11	.0283	.205	622	516	441	385	342	307	279	255	236	219	204	191	180	170	161
9-#11	.0318	.211	653	539	459	399	354	317	288	264	243	225	210	196	185	174	165
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—22″ x 22″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0114	.162	519	446	393	350	315	287	264	244	226	212	198	187	177	167	159
8-#8	.0131	.165	534	459	401	358	322	293	268	249	230	215	202	190	179	170	162
9-#8	.0147	.168	550	471	412	366	329	299	275	253	234	219	205	194	182	172	164
10-#8	.0163	.172	566	484	423	375	335	305	278	257	240	222	208	195	185	175	166
11-#8	.0179	.175	582	495	431	382	343	312	284	263	242	227	212	199	188	178	169
12-#8	.0196	.179	598	503	440	390	350	315	290	265	245	228	213	202	190	180	170
6-#9	.0124	.164	528	453	397	353	320	290	266	246	228	213	200	188	178	169	160
7-#9	.0145	.168	548	469	411	364	328	297	273	253	234	218	203	192	182	172	164
8-#9	.0165	.172	568	485	422	375	336	304	280	257	240	223	208	196	185	176	167
9-#9	.0186	.176	588	500	435	383	345	312	286	263	243	227	212	200	189	179	170
10-#9	.0206	.181	608	513	446	393	352	318	292	268	248	232	216	202	192	181	172
11-#9	.0227	.185	628	530	457	404	361	326	297	273	252	236	220	207	195	184	175
12-#9	.0248	.189	648	545	470	415	370	333	304	280	258	239	223	211	198	187	178
6-#10	.0157	.173	560	479	416	368	332	300	275	253	235	219	205	193	182	173	164
7-#10	.0183	.176	586	500	435	385	344	312	285	263	243	227	212	201	189	179	170
8-#10	.0210	.182	611	509	442	390	350	315	288	266	246	228	213	200	190	180	170
9-#10	.0236	.187	637	535	463	407	363	329	300	275	254	237	222	208	196	185	175
10-#10	.0262	.192	662	555	480	422	376	337	308	282	262	243	227	212	200	189	180
11-#10	.0289	.199	687	573	490	430	384	342	313	287	265	246	229	215	202	191	181
6-#11	.0193	.178	595	508	440	389	348	315	290	266	247	229	214	201	190	180	170
7-#11	.0226	.185	626	530	457	403	360	325	297	273	252	235	212	207	195	184	174
8-#11	.0258	.191	658	551	475	417	373	337	307	281	261	242	226	211	200	189	179
9-#11	.0290	.199	689	572	491	433	385	345	314	287	266	247	230	216	203	192	182
10-#11	.0323	.203	720	600	512	448	398	360	326	298	275	256	237	223	210	198	188
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20,000$ psi

COLUMN SIZE—23″ x 23″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0105	.152	557	483	427	382	346	316	291	269	251	235	221	208	197	187	178
8-#8	.0120	.154	572	495	438	391	354	323	297	275	256	240	225	212	201	190	181
9-#8	.0134	.157	588	508	447	400	361	329	303	280	260	244	229	215	204	193	184
10-#8	.0150	.159	604	521	458	409	369	336	309	286	266	248	233	219	208	197	187
11-#8	.0164	.162	620	534	468	417	376	342	314	290	270	252	237	223	210	199	190
12-#8	.0179	.165	636	546	478	425	383	348	320	295	274	256	240	226	213	202	192
6-#9	.0114	.153	566	491	433	388	351	321	295	273	254	238	223	211	199	189	180
7-#9	.0132	.157	586	506	446	398	360	328	301	279	260	242	228	215	203	193	183
8-#9	.0152	.159	606	523	460	410	370	338	310	287	266	249	234	220	208	197	188
9-#9	.0170	.163	626	538	472	420	379	345	317	292	271	254	238	224	212	201	191
10-#9	.0189	.167	646	554	484	430	388	352	323	298	276	258	242	227	215	203	193
11-#9	.0208	.171	666	569	496	440	395	359	329	303	281	262	245	231	218	206	196
12-#9	.0227	.175	686	584	508	450	403	366	334	308	286	266	249	234	221	209	199
6-#10	.0144	.159	598	516	453	405	365	333	306	283	263	246	231	217	205	195	185
7-#10	.0168	.163	624	536	471	419	378	344	316	291	271	253	237	224	211	200	190
8-#10	.0192	.166	649	556	487	433	390	354	325	299	279	260	244	230	217	205	195
9-#10	.0217	.172	675	575	502	445	400	363	332	306	284	264	248	233	220	209	198
10-#10	.0240	.176	700	595	518	458	411	372	340	313	291	271	253	238	225	213	202
11-#10	.0264	.179	725	615	534	472	422	382	349	322	298	277	260	244	230	218	207
6-#11	.0177	.164	633	544	476	424	382	348	319	295	273	256	240	226	213	202	192
7-#11	.0207	.170	664	567	496	440	395	359	328	303	281	262	246	231	218	207	196
8-#11	.0237	.175	696	592	515	456	410	371	339	313	290	270	253	238	224	212	202
9-#11	.0266	.179	727	617	536	473	423	383	350	322	299	278	260	245	231	219	207
10-#11	.0295	.184	758	640	554	488	436	395	360	331	306	285	267	251	236	224	212
11-#11	.0324	.188	789	664	573	502	450	406	371	341	315	293	274	257	242	229	218
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\quad$ $f_s = 20{,}000$ psi

COLUMN SIZE—24″ x 24″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#7	.0104	.145	606	529	469	422	383	351	324	300	280	263	247	236	221	210	200
12-#7	.0125	.149	630	548	485	436	394	361	334	310	288	269	253	239	226	215	204
8-#8	.0110	.146	612	534	474	426	387	354	327	300	282	264	249	235	223	211	201
10-#8	.0137	.151	644	560	495	444	402	367	338	313	294	273	257	242	229	217	207
12-#8	.0164	.155	676	585	516	461	417	381	350	324	302	282	265	250	236	224	213
6-#9	.0104	.145	606	529	470	422	384	351	324	301	281	263	247	233	221	210	200
8-#9	.0139	.151	646	562	497	445	403	368	339	314	293	274	257	243	230	218	208
10-#9	.0174	.157	686	592	522	466	422	384	354	327	304	285	267	252	238	226	214
11-#9	.0191	.160	706	608	535	477	431	392	360	333	310	289	272	256	242	229	218
12-#9	.0208	.164	726	624	547	487	438	400	366	338	314	294	275	259	245	232	220
14-#9	.0243	.169	766	656	572	508	456	416	380	351	326	304	284	268	253	240	227
6-#10	.0132	.150	638	556	491	441	399	365	336	312	290	272	255	241	228	217	206
7-#10	.0154	.154	664	575	507	454	411	375	345	320	298	278	261	246	233	221	211
8-#10	.0176	.157	689	597	526	469	423	386	355	329	306	286	268	253	239	227	215
9-#10	.0198	.163	715	614	539	481	432	394	362	334	311	290	272	256	242	229	218
10-#10	.0220	.165	740	636	556	495	446	406	372	344	319	298	279	263	249	235	224
12-#10	.0265	.172	791	675	588	522	468	425	389	359	333	310	291	274	258	245	232
13-#10	.0286	.176	816	694	603	534	479	434	397	365	339	316	296	278	262	248	236
6-#11	.0162	.155	673	583	514	460	416	379	349	323	301	281	264	249	236	223	212
7-#11	.0190	.160	704	606	534	476	429	391	359	332	308	289	271	255	241	229	217
8-#11	.0217	.165	736	632	554	493	443	403	370	341	317	296	278	262	247	234	222
9-#11	.0244	.169	767	657	573	509	457	416	381	351	326	304	285	268	254	240	228
10-#11	.0271	.173	798	681	594	526	472	428	392	361	335	313	293	275	260	246	233
11-#11	.0298	.177	829	705	613	542	486	440	402	370	343	320	300	281	265	251	239
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—25″ x 25″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#7	.0115	.141	671	588	523	471	429	393	363	337	315	296	278	263	249	236	225
8-#8	.0101	.139	653	573	511	461	419	385	356	331	309	290	273	258	245	233	222
10-#8	.0126	.143	685	600	532	479	436	399	368	342	319	299	282	266	252	239	228
12-#8	.0151	.147	717	625	554	497	451	412	381	353	330	308	290	274	259	246	233
6-#9	.0096	.139	647	568	506	456	415	381	352	328	306	287	271	256	242	230	220
8-#9	.0128	.144	687	600	534	480	436	400	369	342	319	299	282	266	252	239	228
10-#9	.0160	.149	727	633	560	502	456	417	384	355	332	310	292	276	261	248	235
11-#9	.0176	.151	747	650	573	513	465	425	392	364	338	317	298	281	266	252	240
12-#9	.0192	.154	767	663	587	524	475	433	399	369	344	321	302	285	269	255	243
14-#9	.0224	.159	807	697	613	547	494	450	413	382	356	332	312	294	277	263	250
6-#10	.0122	.142	679	594	528	476	433	397	367	340	318	298	280	265	251	238	227
7-#10	.0142	.145	705	615	546	490	446	409	377	350	326	306	288	272	257	244	233
8-#10	.0162	.149	730	635	562	504	457	418	385	357	333	311	293	276	262	248	236
9-#10	.0184	.153	756	655	580	518	469	428	394	365	340	318	299	282	267	253	240
10-#10	.0203	.156	781	676	598	532	481	438	402	373	347	324	305	287	272	258	245
11-#10	.0223	.159	806	696	612	546	492	449	412	381	354	331	311	293	277	263	250
12-#10	.0244	.165	832	714	625	556	501	456	418	386	359	335	314	295	279	265	252
13-#10	.0264	.165	857	736	644	573	516	469	430	397	369	345	323	304	287	272	259
6-#11	.0150	.147	714	623	552	495	449	411	379	352	328	307	289	272	258	245	233
7-#11	.0174	.151	745	647	572	512	464	424	390	362	337	316	296	280	265	251	239
8-#11	.0200	.156	777	672	592	530	478	436	401	372	345	323	304	286	271	257	244
9-#11	.0225	.159	808	697	613	547	494	450	413	382	355	332	312	294	278	263	250
10-#11	.0250	.163	839	721	633	563	508	462	424	391	364	340	319	300	284	269	255
11-#11	.0274	.165	870	747	654	582	524	476	437	403	375	350	328	309	292	276	263
12-#11	.0300	.170	901	770	672	596	536	487	446	411	381	356	333	314	296	280	266
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—26″ x 26″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#8	.0105	.135	712	627	561	507	463	425	393	366	342	322	303	286	272	258	246
11-#8	.0128	.138	744	655	583	526	479	440	407	379	353	332	313	296	280	266	254
13-#8	.0152	.141	775	679	605	545	495	455	420	390	364	342	322	304	288	274	261
15-#8	.0175	.145	807	705	625	563	511	468	432	401	374	350	330	311	295	280	266
7-#9	.0103	.134	710	626	560	506	462	425	394	366	343	321	304	287	272	259	247
8-#9	.0118	.136	730	643	574	519	473	435	402	374	350	328	309	293	278	264	251
9-#9	.0133	.138	750	660	588	530	483	444	410	382	356	334	315	298	283	269	256
10-#9	.0148	.141	770	675	600	541	493	452	418	388	362	340	320	302	286	272	259
11-#9	.0163	.143	790	690	615	553	502	460	425	394	368	346	325	307	291	276	263
12-#9	.0177	.146	810	707	626	564	511	468	433	400	374	351	330	311	295	280	266
13-#9	.0192	.148	830	724	641	575	521	477	440	409	380	356	335	316	300	284	270
15-#9	.0222	.152	870	755	667	598	541	495	455	421	394	368	346	326	308	293	278
6-#10	.0113	.135	722	636	569	514	469	431	399	371	347	326	307	290	276	262	250
8-#10	.0150	.141	773	677	604	544	495	454	419	389	364	341	321	303	287	273	260
10-#10	.0188	.148	824	718	635	570	512	474	436	405	377	353	332	313	297	282	268
12-#10	.0226	.153	875	759	670	600	543	495	456	423	393	368	346	326	309	292	278
14-#10	.0263	.159	926	800	703	627	566	516	474	439	408	381	358	337	318	302	287
6-#11	.0138	.139	757	665	593	535	486	446	413	384	358	336	316	299	284	270	257
7-#11	.0161	.143	788	688	612	551	500	460	425	393	367	344	324	306	290	276	262
8-#11	.0184	.147	820	715	634	569	516	472	435	404	377	352	332	314	296	282	268
9-#11	.0208	.151	851	740	654	586	530	485	447	415	386	362	339	320	303	287	274
10-#11	.0230	.154	882	765	675	604	546	499	458	425	396	370	348	328	310	294	280
11-#11	.0254	.157	913	790	695	620	560	511	470	435	405	379	355	335	316	300	286
13-#11	.0300	.163	976	840	736	656	591	538	494	456	424	396	371	350	331	314	298
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—27″ x 27″

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
14-#8	.0153	.136	836	736	657	594	541	497	460	428	400	375	354	335	317	302	287
16-#8	.0174	.140	868	761	678	611	556	510	472	438	409	384	361	342	324	308	293
18-#8	.0196	.143	899	786	699	629	572	524	483	449	419	393	370	349	331	314	299
10-#9	.0137	.134	815	718	642	581	530	488	451	420	393	369	348	329	312	297	283
12-#9	.0165	.138	855	751	670	605	551	506	467	435	406	381	359	340	322	306	291
14-#9	.0193	.143	895	783	696	626	569	521	481	447	417	391	368	347	329	313	298
16-#9	.0220	.147	935	815	722	649	588	538	496	461	430	402	378	357	338	321	306
6-#10	.0150	.129	767	679	610	553	506	466	432	403	377	354	335	317	301	286	273
7-#10	.0122	.131	793	700	628	569	520	479	444	413	386	364	343	325	308	293	280
8-#10	.0139	.134	818	721	645	584	532	490	453	422	394	370	349	330	313	298	284
9-#10	.0158	.138	844	741	662	597	544	498	462	429	401	376	354	335	318	302	288
10-#10	.0174	.140	869	762	679	612	557	511	472	438	410	384	362	342	324	308	293
11-#10	.0192	.142	894	783	696	627	570	523	482	448	418	392	369	348	330	314	299
12-#10	.0209	.146	920	803	711	640	580	532	490	455	424	398	374	353	334	317	302
13-#10	.0227	.147	945	824	730	654	595	544	502	466	433	406	382	361	342	324	309
14-#10	.0243	.150	971	844	747	670	607	554	511	474	442	413	386	366	347	329	316
15-#10	.0262	.152	996	865	764	684	619	566	521	482	449	420	394	372	352	334	318
6-#11	.0128	.133	802	708	634	573	523	481	446	415	388	365	344	325	309	294	280
8-#11	.0172	.139	865	759	677	610	556	510	471	438	409	384	362	342	324	308	293
9-#11	.0193	.142	896	785	698	628	571	524	483	449	419	393	370	349	331	314	300
10-#11	.0214	.144	927	810	719	647	588	539	497	462	430	404	380	358	340	322	307
11-#11	.0236	.149	958	833	738	662	600	549	505	468	437	409	384	363	343	326	310
12-#11	.0257	.152	989	859	758	679	615	562	517	479	446	417	392	370	350	332	316
13-#11	.0277	.156	1021	884	779	696	629	574	528	488	454	425	399	376	355	337	321
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—28" x 28"

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#8	.0101	.124	819	729	657	596	548	505	470	439	411	387	366	346	329	314	299
12-#8	.0121	.127	851	756	679	616	565	520	483	451	423	397	375	356	337	321	307
14-#8	.0141	.130	882	780	700	635	580	535	496	462	433	406	384	363	344	328	313
8-#9	.0102	.124	821	730	659	598	549	507	470	440	412	388	366	347	330	314	300
10-#9	.0128	.129	861	764	685	621	568	524	485	452	424	398	376	356	338	322	306
12-#9	.0153	.132	901	796	713	645	590	543	503	468	439	412	388	368	348	332	316
14-#9	.0179	.136	941	829	740	670	610	560	518	482	450	423	399	377	358	340	324
7-#10	.0113	.125	839	746	671	610	560	517	480	448	420	395	373	354	336	320	305
8-#10	.0130	.129	864	766	688	624	570	525	486	454	425	399	378	358	339	323	308
9-#10	.0146	.131	890	787	705	639	584	538	498	465	435	409	386	364	346	329	314
10-#10	.0162	.133	915	807	724	655	597	550	510	474	444	417	393	371	353	336	320
11-#10	.0178	.136	940	828	739	668	609	560	518	481	450	422	399	377	357	340	324
12-#10	.0194	.137	966	850	759	685	625	574	530	494	461	433	408	385	365	347	331
13-#10	.0211	.140	991	870	775	698	635	583	539	500	467	438	413	390	370	352	335
14-#10	.0227	.143	1017	890	790	712	646	593	548	508	474	446	419	396	375	356	338
6-#11	.0120	.127	848	755	676	614	564	519	481	450	421	395	374	354	336	320	306
7-#11	.0139	.130	879	778	698	631	578	532	494	460	430	405	382	362	343	326	312
8-#11	.0159	.133	911	804	720	652	595	547	507	472	442	415	391	370	352	334	318
9-#11	.0179	.136	942	830	740	670	610	560	519	482	451	423	399	378	358	340	324
10-#11	.0199	.139	973	855	762	688	625	575	531	494	461	433	407	385	365	347	330
11-#11	.0219	.142	1004	880	782	705	641	587	542	504	470	441	415	392	371	353	336
12-#11	.0239	.145	1035	905	802	721	655	600	554	514	480	449	423	400	378	359	342
13-#11	.0259	.147	1067	930	825	740	673	615	567	526	490	459	432	408	386	367	349
14-#11	.0318	.154	1098	951	840	750	680	620	570	529	492	460	432	408	386	366	348
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—29″ x 29″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
19-#8	.0178	.129	1011	896	804	729	667	614	570	532	497	468	442	418	397	378	360
21-#8	.0198	.132	1042	921	824	746	682	628	582	542	507	477	449	425	403	384	366
11-#9	.0131	.123	930	828	746	679	623	576	535	499	469	442	417	395	376	358	341
12-#9	.0143	.124	950	846	761	692	634	586	545	508	477	449	424	404	382	363	347
13-#9	.0154	.125	970	862	776	705	646	597	554	517	485	456	431	408	388	369	353
14-#9	.0166	.126	990	878	789	718	658	607	564	526	493	464	438	415	394	376	356
15-#9	.0178	.129	1010	896	804	729	666	614	570	531	497	467	441	418	397	378	360
16-#9	.0190	.131	1030	911	816	739	675	622	576	537	502	472	445	421	400	381	364
17-#9	.0202	.132	1050	926	830	751	686	632	585	547	511	480	452	428	406	386	368
18-#9	.0215	.134	1070	944	844	763	696	640	593	553	517	486	458	433	411	391	373
8-#10	.0120	.121	913	814	735	670	615	569	529	494	464	437	413	392	372	355	339
9-#10	.0136	.123	938	833	752	685	629	581	540	504	473	445	420	398	379	361	344
10-#10	.0151	.125	964	857	771	701	643	593	551	514	482	454	428	406	386	367	351
11-#10	.0166	.127	989	877	789	716	656	605	661	523	491	461	435	413	392	373	356
12-#10	.0184	.130	1015	899	806	730	668	616	571	532	498	468	442	418	397	377	360
13-#10	.0196	.132	1040	919	824	719	681	627	580	541	506	476	448	424	403	383	366
14-#10	.0210	.133	1065	940	842	762	698	640	592	552	516	485	458	432	410	391	372
15-#10	.0232	.136	1091	960	858	775	708	650	601	559	523	491	463	437	415	395	376
16-#10	.0242	.137	1116	882	877	792	722	663	613	570	533	500	471	446	422	401	383
8-#11	.0148	.125	960	853	768	698	640	591	549	512	480	452	427	404	384	366	349
9-#11	.0167	.127	991	879	791	717	657	606	562	525	491	462	437	413	392	374	357
10-#11	.0186	.130	1022	908	812	736	673	620	574	535	501	471	444	421	399	380	362
11-#11	.0205	.133	1054	930	832	754	688	633	586	546	510	480	452	428	406	386	368
12-#11	.0223	.136	1085	955	854	771	704	646	598	556	520	488	460	435	412	392	374
13-#11	.0240	.137	1116	982	877	791	722	663	613	570	533	500	471	446	422	401	383
14-#11	.0260	.140	1147	1006	896	808	736	675	624	580	542	508	478	452	428	407	388
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—30″ x 30″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#9	.0100	.115	940	844	765	699	644	597	556	520	490	462	437	415	395	377	360
11-#9	.0122	.117	980	879	795	725	668	618	575	540	506	477	452	428	408	388	372
13-#9	.0144	.120	1020	911	822	750	690	637	593	554	520	490	464	440	418	398	380
15-#9	.0167	.123	1060	944	850	775	710	656	610	570	535	504	475	451	428	409	389
8-#10	.0113	.117	963	863	781	713	657	609	566	530	498	470	445	421	400	382	366
9-#10	.0127	.118	989	885	800	730	671	621	580	541	509	479	454	430	410	390	373
10-#10	.0141	.119	1014	907	820	748	686	636	591	553	520	490	463	439	417	398	380
11-#10	.0155	.122	1039	925	835	760	699	645	600	560	526	495	468	443	421	401	384
12-#10	.0169	.123	1065	950	855	779	714	660	614	572	537	505	478	453	430	411	391
13-#10	.0183	.125	1090	970	872	793	726	670	623	581	545	513	484	459	436	415	396
14-#10	.0198	.128	1116	990	890	807	738	681	632	590	551	520	490	464	441	419	400
15-#10	.0212	.129	1141	1011	909	825	754	685	644	600	563	528	498	473	448	427	406
16-#10	.0226	.131	1166	1032	925	837	765	705	654	609	570	536	505	478	454	431	412
17-#10	.0240	.133	1192	1050	944	854	778	716	664	618	578	544	512	484	460	438	416
6-#11	.0104	.115	947	850	770	704	649	600	560	524	493	465	440	418	398	380	363
7-#11	.0121	.117	978	876	793	724	666	617	575	539	505	477	451	428	406	388	372
8-#11	.0139	.119	1010	904	816	745	685	634	590	550	517	488	461	438	416	396	380
9-#11	.0156	.122	1041	928	838	762	700	646	600	561	527	497	469	444	422	402	384
10-#11	.0173	.124	1072	953	860	780	716	661	615	575	537	506	478	453	430	411	392
11-#11	.0190	.126	1103	980	880	801	734	677	629	586	550	518	488	463	440	419	399
12-#11	.0208	.129	1134	1005	901	819	749	690	640	596	559	525	495	470	445	425	404
13-#11	.0225	.131	1166	1030	925	838	765	705	654	609	570	536	505	478	454	431	412
14-#11	.0242	.133	1197	1056	945	856	780	723	666	620	580	545	514	485	462	438	418
15-#11	.0260	.135	1228	1081	966	875	796	733	679	631	590	555	523	494	469	446	425
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—31″ x 31″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
16-#8	.0132	.115	1064	954	865	791	728	675	629	589	554	523	495	470	447	426	408
20-#8	.0165	.118	1127	1007	912	833	766	709	660	617	580	546	517	491	467	445	425
12-#9	.0125	.115	1051	944	855	782	720	668	622	582	548	517	489	464	442	422	403
14-#9	.0146	.116	1091	978	888	810	748	691	643	602	566	534	505	480	456	435	416
16-#9	.0167	.118	1131	1012	915	836	769	712	662	620	582	548	519	493	469	446	426
18-#9	.0188	.122	1171	1044	942	857	787	728	676	632	592	559	528	500	475	453	433
20-#9	.0208	.125	1211	1076	969	881	808	746	692	646	606	570	538	510	485	462	441
8-#10	.0105	.112	1014	912	829	759	700	650	606	569	535	505	478	454	433	413	395
10-#10	.0132	.115	1065	956	866	792	730	676	630	590	555	524	495	470	443	427	418
11-#10	.0145	.116	1090	977	885	809	745	690	643	602	566	534	505	479	456	435	415
12-#10	.0159	.118	1116	999	903	825	758	702	654	611	574	541	512	486	462	440	421
13-#10	.0172	.118	1141	1010	923	844	776	718	668	625	587	554	524	497	473	450	430
14-#10	.0185	.120	1166	1040	940	857	788	729	678	634	595	560	530	502	478	456	435
15-#10	.0199	.124	1192	1061	955	869	777	736	684	638	598	564	532	504	479	456	436
16-#10	.0212	.125	1217	1081	974	886	812	749	696	649	609	573	541	513	487	464	443
17-#10	.0225	.127	1243	1102	992	900	824	760	705	658	617	580	548	519	492	469	448
18-#10	.0238	.128	1268	1124	1010	917	839	774	718	669	627	589	556	527	500	476	454
8-#11	.0130	.115	1061	952	863	789	727	674	628	588	553	522	494	469	446	425	413
9-#11	.0146	.116	1092	974	879	801	736	680	632	591	555	523	494	468	445	424	410
10-#11	.0162	.118	1123	1004	908	830	763	706	657	615	578	544	515	489	465	443	423
11-#11	.0178	.120	1155	1031	932	850	781	722	672	628	590	556	525	498	473	451	431
12-#11	.0195	.123	1186	1056	951	867	795	735	683	637	598	563	532	503	479	457	436
13-#11	.0210	.124	1217	1093	976	887	814	752	698	652	611	575	543	515	489	466	445
14-#11	.0228	.127	1248	1106	996	904	828	764	709	661	619	582	550	521	494	471	450
15-#11	.0244	.129	1279	1132	1016	922	843	777	721	672	629	592	558	529	502	478	456
16-#11	.0260	.130	1311	1160	1040	943	863	795	737	686	643	604	570	539	512	487	465
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $\quad f_s = 20{,}000$ psi

COLUMN SIZE—32″ x 32″

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#9	.0117	.110	1104	995	905	830	766	712	665	624	588	555	526	500	476	455	435
14-#9	.0137	.112	1144	1029	935	857	791	734	685	642	604	570	540	514	488	466	446
16-#9	.0156	.114	1184	1062	965	882	812	754	704	660	619	584	553	525	500	476	456
18-#9	.0176	.117	1224	1097	992	906	835	773	720	675	633	596	565	535	509	485	465
20-#9	.0195	.120	1264	1130	1020	930	855	790	735	687	645	608	575	545	518	494	472
10-#10	.0124	.110	1118	1008	916	841	776	721	674	631	595	562	532	506	482	460	440
11-#10	.0136	.112	1143	1030	935	855	790	733	684	640	603	569	540	513	487	466	445
12-#10	.0149	.113	1169	1050	954	874	805	746	696	651	614	580	549	520	496	473	452
13-#10	.0161	.114	1194	1071	972	890	820	760	709	665	624	589	558	530	504	480	460
14-#10	.0173	.115	1220	1095	991	907	835	775	721	675	635	600	567	539	512	489	468
15-#10	.0186	.118	1245	1115	1009	920	845	784	730	682	640	604	571	543	516	492	470
16-#10	.0198	.120	1270	1134	1024	935	859	794	738	690	648	610	578	548	520	496	474
17-#10	.0211	.121	1296	1155	1042	950	875	808	751	702	659	621	586	556	529	504	482
18-#10	.0224	.123	1321	1178	1060	966	885	819	760	710	666	627	593	562	535	510	485
8-#11	.0122	.110	1114	1005	914	839	774	719	671	630	593	560	531	505	480	459	439
9-#11	.0137	.112	1145	1030	935	857	792	734	685	642	604	570	540	514	488	467	446
10-#11	.0152	.113	1176	1055	960	880	810	751	701	656	617	584	552	524	500	476	455
11-#11	.0167	.115	1207	1081	980	898	826	766	714	669	629	594	561	533	507	484	463
12-#11	.0183	.117	1238	1109	1002	916	845	781	726	681	640	604	570	541	515	491	470
13-#11	.0198	.120	1270	1133	1023	934	858	795	739	690	648	610	577	547	520	496	474
14-#11	.0213	.121	1301	1160	1050	955	878	811	755	705	662	623	590	558	531	506	484
15-#11	.0228	.124	1332	1186	1070	970	890	822	764	714	669	630	595	563	535	510	486
16-#11	.0244	.125	1363	1211	1090	991	910	840	780	728	682	641	606	575	545	520	496
For f_s = 16,000 psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

SPIRALLY REINFORCED SQUARE COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—33″ x 33″

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
13-#9	.0119	.106	1179	1065	972	895	826	770	720	676	639	603	572	544	519	496	474
15-#9	.0138	.108	1219	1100	1002	920	850	791	740	695	654	618	586	556	531	506	485
17-#9	.0156	.111	1259	1132	1030	944	871	809	755	709	666	630	596	565	540	515	493
19-#9	.0175	.113	1299	1168	1060	971	894	830	774	725	681	645	610	578	552	526	503
10-#10	.0117	.106	1173	1060	970	892	825	768	719	675	636	601	570	542	517	495	473
11-#10	.0128	.107	1198	1082	986	906	840	780	730	685	645	610	579	551	524	501	480
12-#10	.0140	.108	1224	1106	1010	930	860	800	749	703	664	626	595	565	539	515	493
13-#10	.0152	.110	1249	1124	1022	940	866	805	751	705	664	628	595	565	538	514	492
14-#10	.0163	.111	1275	1148	1042	956	884	820	755	718	676	639	605	574	547	521	500
15-#10	.0175	.113	1300	1168	1060	971	895	830	775	725	682	646	610	579	552	526	504
16-#10	.0187	.115	1325	1190	1078	985	909	842	785	735	690	651	616	585	557	531	508
17-#10	.0198	.116	1351	1211	1098	1003	925	856	798	746	703	662	626	595	565	540	515
18-#10	.0210	.117	1376	1232	1115	1019	939	869	808	758	711	671	635	601	572	545	522
19-#10	.0221	.118	1402	1258	1135	1037	951	882	821	768	721	679	643	611	581	554	530
8-#11	.0115	.106	1169	1058	962	886	820	763	714	670	632	597	566	539	514	491	470
9-#11	.0129	.107	1200	1084	989	909	841	782	731	687	646	612	580	552	525	503	481
10-#11	.0143	.109	1231	1111	1012	930	858	796	745	699	658	621	590	560	534	510	488
11-#11	.0157	.111	1262	1138	1031	948	875	813	758	711	670	632	599	568	542	517	495
12-#11	.0172	.112	1293	1162	1058	970	895	830	774	725	682	644	610	580	551	527	504
13-#11	.0186	.115	1325	1190	1078	985	909	842	785	735	690	651	616	585	557	531	508
14-#11	.0201	.116	1356	1215	1100	1008	926	859	800	748	704	663	628	596	566	541	516
15-#11	.0215	.117	1387	1241	1122	1025	945	875	815	762	716	675	639	605	575	550	526
16-#11	.0229	.120	1418	1268	1143	1042	958	886	825	770	724	681	645	611	581	554	529
17-#11	.0244	.121	1450	1292	1168	1064	976	904	840	785	738	695	656	622	592	564	539
For $f_s = 16{,}000$ psi multiply by				.935	.940	.950	.960	.965	.970	.970	.970	.975	.975	.980	.985	.985	.985

Outside diameter of spiral should be 3 in. less than side of column.

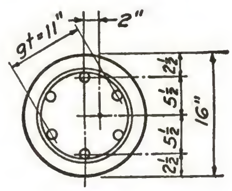
ECCENTRICALLY LOADED CONCRETE COLUMNS
SECTION III—SPIRALLY REINFORCED ROUND COLUMNS

This third section covers eccentric loads on spirally reinforced round concrete columns and parallels exactly the first section, so the explanation on pages 275-280 and 297-298 should be read before going on with the following description.

The necessary size and pitch of spiral reinforcement can be taken from the tables for axially loaded spirally reinforced round columns on pages 252 to 257, inclusive. The vertical bars are spaced uniformly around a ring just inside of and in contact with the spiral.

While the scope of these tables is sufficient for most purposes, some illustrative examples are shown for those who wish to design beyond their range or merely to see how they were prepared.

Example—For the table on page 337, verify the value $N = 135$ with an eccentricity of 2 in. for a 16 in. round spirally reinforced column of 3000 psi concrete reinforced with 6-#8 bars, using $f_s = 20{,}000$ psi, $f_c = 675$ psi, $n = 10$.



6-#8 Bars = 4.74 sq in. ⇒ 42.7 sq in.

Since the point of application is within R/4 of the center of the 16 in. round section, the load apparently acts within the kern of the transformed section,* producing compression over the entire area. Solve by the elastic theory using the transformed section:—

Section	*Area of Transformed Section*		*Capacity (Axial Loading)*		*Moment of Inertia*	
Concrete	$\frac{\pi}{4}(16)^2$	= 201.0	201 @ 675	= 135,700	$201 \times \frac{8^2}{4}$	= 3216
Bars	$9 \times 6 \times 0.79$	= 42.7	6×0.79 @ 20,000	= 94,800	† $42.7 \times \frac{(5.5)^2}{2}$	= 646
		243.7 sq in.		230,500 lb		3862 in.^{4}

$$\text{Unit direct stress} = \frac{N}{A} = \frac{135{,}000}{243.7} = 554 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = \frac{135{,}000 \times 2 \times 8}{3862} = 559 \text{ psi}$$

1113 psi Max Comp

5 psi possible tension, showing that the load is really just outside of the kern of the section.

$f_s = 10\,(1113 - 2.5 \times 1118/16) = 9380$ psi comp

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* See page 93.

† The transformed steel area is assumed to be a ring with a mean radius r_m of 5.5 in.

$$I = \frac{\pi}{64}(D_1^4 - D_2^4) = \frac{Ar_m^2}{2}$$

ECCENTRICALLY LOADED SPIRALLY REINFORCED ROUND CONCRETE COLUMNS

The allowable combined stress for this condition is obtained by using the same procedure as on page 277 viz., $\frac{f_a}{F_a} + \frac{f_b}{F_b} = 1$:—

$$\frac{135{,}000}{230{,}500} + \frac{559}{1350} = 0.585 + 0.414 = 0.999.$$

By the method of the 1951 code, using the simplified procedure of Ex. I—Second Solution, page 278:—$f_a = \frac{230{,}500}{243.7} = 946$ psi $\quad C = 946/1350 = 0.701$

$$D = \frac{At^2}{2I} = \frac{243.7 \times 16 \times 16}{2 \times 3862} = 8.08$$

$$\frac{B}{t} = \frac{CD}{t} = \frac{0.701 \times 8.08}{16} = 0.354 \qquad N = \frac{P}{1 + \frac{Be}{t}} = \frac{230{,}500}{(1 + 0.354 \times 2)} = 135 \text{ kips}$$

The above value for D can also be obtained:— $\quad p_g = \frac{6 \times 0.79}{201} = 0.0236;$

$$g = \frac{11}{16} = 0.688$$

$$D\,* = \frac{1 + (n-1)\,p_g}{\frac{1}{8} + 0.25\,(n-1)\,p_g g^2} = \frac{1 + 0.212}{0.125 + 0.25 \times 0.212 \times (0.688)^2} = 8.08$$

When the load falls outside of the kern of the section and the ratio $e/t \lesseqgtr 2/3$, as explained on page 279, ACI 1109a permits design for an uncracked section.†

Example—Show that using the data from the previous example but increasing the eccentricity to 6 in. reduces the value of N to 74 kips. Taking the values established in the previous example gives:—

$$\text{Unit direct stress} = \frac{N}{A} = \frac{74{,}000}{243.7} = 304 \text{ psi}$$

$$\text{Unit bending stress} = \frac{Nec}{I} = \frac{74{,}000 \times 6 \times 8}{3862} = 920 \text{ psi}$$

1224 psi Max Comp
616 psi Max Tens

$$N = \frac{P}{1 + \frac{Be}{t}} = \frac{230{,}500}{1 + 0.354 \times 6} = 74 \text{ kips}$$

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* See page 278.

† If tension is neglected in the concrete on the side of the neutral axis opposite from the load (theory of the cracked section), then it is necessary to balance moments about the point of application of the load and also to balance forces perpendicular to the cross section of the column as illustrated on pages 279-280. With the vertical reinforcement arranged in a ring around the column, the resulting cubic and trigonometric equations are rather involved. (See Sutherland and Reese "Reinforced Concrete Design," John Wiley & Sons, Inc., 1943, pages 124-125.) This is beyond the scope of a handbook such as this and properly belongs in a textbook on the subject.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—14 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#5	.0121	.355	141	104	82	68	58	51	45	40	37	34	19	17	16	14	13
8-#5	.0161	.375	154	112	88	72	62	54	47	42	38	35	22	20	18	16	15
10-#5	.0201	.395	166	119	93	76	64	56	49	44	40	36	24	22	20	18	17
6-#6	.0171	.382	157	114	89	73	62	54	48	43	39	35	22	20	18	16	15
7-#6	.0200	.395	166	119	93	76	64	56	49	44	40	36	24	22	20	18	17
8-#6	.0228	.408	174	124	96	78	66	57	50	45	41	37	26	23	21	20	18
9-#6	.0257	.421	183	129	99	81	68	59	52	46	42	38	27	25	22	21	19
6-#7	.0234	.411	176	125	97	79	66	58	51	45	41	37	26	23	21	19	18
7-#7	.0273	.428	188	132	101	82	69	60	53	47	42	39	28	25	23	21	19
8-#7	.0312	.445	200	138	106	86	72	62	54	49	44	40	30	27	25	23	21
6-#8	.0308	.442	199	138	106	86	72	62	54	49	44	40	30	27	25	23	21
7-#8	.0359	.463	215	147	112	90	75	65	57	51	46	42	32	29	27	25	23
8-#8	.0410	.482	230	155	117	94	78	67	59	52	47	43	35	32	29	27	25
6-#9	.0390	.475	224	152	115	92	77	66	58	52	47	42	34	31	28	26	24
7-#9	.0454	.498	244	163	122	98	82	70	61	54	49	44	37	34	31	28	26
6-#10	.0495	.508	256	170	127	101	84	72	63	56	50	46	39	35	32	30	28
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	†	†	†	†	†

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—16 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0131	.311	189	144	116	98	84	74	66	60	54	50	46	29	26	24	22
9-#6	.0197	.337	215	161	128	107	92	80	71	64	58	53	49	35	32	29	27
6-#7	.0179	.331	208	156	125	104	89	78	70	63	57	52	48	33	30	28	26
7-#7	.0209	.342	220	164	130	108	93	81	72	65	59	54	50	36	33	30	28
9-#7	.0269	.364	244	179	141	117	99	86	77	69	62	57	53	41	38	34	32
11-#7	.0328	.382	268	194	152	125	106	92	81	73	66	60	56	45	41	38	35
6-#8	.0236	.352	231	171	135	112	96	84	74	67	60	55	51	38	35	32	29
7-#8	.0275	.366	247	181	142	118	100	87	77	69	63	58	53	41	38	35	32
8-#8	.0314	.378	262	190	149	123	104	91	80	72	65	59	55	44	41	37	35
9-#8	.0353	.390	278	200	156	128	109	94	83	74	67	62	57	47	43	40	37
6-#9	.0298	.373	256	186	147	121	103	89	79	71	64	59	54	43	38	35	32
7-#9	.0348	.388	276	199	155	127	108	94	83	74	67	61	56	47	43	39	37
8-#9	.0398	.404	296	210	164	134	113	98	86	77	70	64	59	51	46	42	39
6-#10	.0379	.398	288	206	160	131	111	96	85	76	69	63	58	50	45	42	39
7-#10	.0443	.416	314	222	171	140	118	102	90	80	73	66	61	54	49	45	42
6-#11	.0466	.422	323	227	175	142	120	104	91	82	74	67	62	55	51	47	43
7-#11	.0544	.441	354	246	188	152	128	110	97	87	78	71	66	60	56	51	48
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	†	†	†	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\quad\quad$ $f_s = 20{,}000$ psi

COLUMN SIZE—18 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0141	.280	244	191	156	133	115	102	91	82	75	69	64	60	56	38	35
9-#7	.0212	.305	280	214	174	146	126	111	99	89	81	75	69	64	60	46	42
11-#7	.0259	.320	304	230	185	155	133	117	104	94	85	78	72	67	63	51	47
6-#8	.0186	.296	267	206	168	141	122	108	96	87	79	73	67	63	59	43	39
7-#8	.0217	.307	283	217	175	147	127	112	100	90	82	75	70	65	60	46	42
8-#8	.0248	.317	298	226	182	153	131	115	103	93	84	77	72	66	62	50	46
9-#8	.0279	.326	314	237	190	159	136	119	106	96	87	80	74	68	64	53	49
11-#8	.0341	.344	346	257	205	170	146	127	113	102	92	84	78	72	67	59	55
6-#9	.0235	.312	292	222	180	151	130	114	102	92	84	77	71	66	62	49	45
7-#9	.0275	.325	312	235	189	158	136	119	106	95	87	80	73	68	64	53	49
8-#9	.0314	.335	332	249	199	166	142	124	110	99	90	83	76	71	66	57	53
9-#9	.0353	.347	352	261	208	172	147	129	114	103	93	85	79	73	68	60	56
10-#9	.0392	.357	372	274	217	180	153	133	118	106	96	88	81	76	70	64	59
6-#10	.0299	.332	324	243	195	162	139	122	108	98	89	81	75	70	65	55	51
7-#10	.0349	.346	350	260	207	172	147	128	114	102	93	85	78	73	68	60	55
8-#10	.0399	.359	375	276	218	181	154	134	119	107	97	89	82	76	71	65	60
9-#10	.0448	.370	401	293	230	190	162	141	125	112	101	93	85	79	74	69	65
6-#11	.0367	.350	359	266	211	175	150	130	116	104	94	86	80	74	69	62	57
7-#11	.0429	.366	390	285	225	186	158	138	122	109	99	91	84	78	72	68	63
8-#11	.0490	.380	422	306	240	197	168	145	129	115	104	96	88	82	76	71	67
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	†	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\quad$ $f_s = 20{,}000$ psi

COLUMN SIZE—20 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0151	.252	307	245	204	175	153	136	122	111	102	94	87	81	76	72	52
7-#8	.0176	.259	323	256	213	182	159	141	126	115	105	97	90	84	79	74	55
8-#8	.0201	.265	338	267	221	188	164	145	130	118	108	100	93	86	81	76	59
9-#8	.0226	.271	354	278	229	194	169	150	134	122	111	102	95	88	83	78	63
10-#8	.0252	.278	370	289	238	201	175	155	139	126	115	106	98	91	85	80	66
11-#8	.0277	.285	386	300	246	208	180	159	142	129	118	108	100	93	87	82	70
12-#8	.0302	.291	402	312	254	215	186	164	147	132	121	111	103	96	90	84	73
6-#9	.0191	.263	332	263	218	186	162	143	129	117	107	99	91	85	80	75	57
7-#9	.0223	.271	352	277	228	194	169	149	134	122	111	102	95	88	83	78	63
8-#9	.0255	.279	372	291	239	202	176	155	139	126	115	106	98	91	86	82	67
9-#9	.0287	.287	392	304	249	210	182	161	144	130	119	109	101	94	88	83	71
10-#9	.0318	.294	412	318	259	219	189	167	149	135	123	113	105	97	91	85	76
11-#9	.0351	.301	432	332	270	227	196	172	154	139	127	116	108	100	94	88	80
6-#10	.0243	.276	364	285	234	199	173	153	137	124	113	104	97	90	84	79	65
7-#10	.0283	.286	390	303	248	210	182	160	144	130	118	109	101	94	88	83	71
8-#10	.0324	.295	415	320	261	220	190	168	150	135	123	113	105	98	91	86	76
9-#10	.0364	.304	441	338	274	231	199	175	156	141	129	118	109	102	95	89	82
10-#10	.0404	.313	466	354	286	240	207	182	162	146	133	122	113	105	98	92	87
6-#11	.0298	.289	399	310	253	214	185	163	146	132	120	111	102	95	89	84	73
7-#11	.0348	.301	430	330	268	226	195	172	153	138	126	116	107	100	93	87	80
8-#11	.0398	.311	462	352	285	239	206	181	161	145	132	122	112	104	98	92	86
9-#11	.0447	.320	493	374	300	252	216	190	169	152	138	127	117	109	102	96	90
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—22 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0146	.226	368	300	253	219	193	172	156	142	131	121	113	105	99	93	88
8-#8	.0166	.232	383	311	261	226	199	177	160	146	134	124	115	108	101	95	90
9-#8	.0187	.238	399	322	270	233	204	182	164	150	137	127	118	110	103	97	92
10-#8	.0208	.243	415	334	279	240	210	187	169	154	141	130	121	113	106	100	94
11-#8	.0229	.248	431	345	288	247	216	192	173	158	144	133	124	116	108	102	96
12-#8	.0250	.253	447	356	297	254	222	197	178	161	148	136	127	118	111	104	98
6-#9	.0158	.230	377	306	258	223	196	175	158	144	133	123	114	107	100	94	89
7-#9	.0184	.237	397	321	269	232	204	182	164	149	137	127	118	110	103	97	92
8-#9	.0211	.244	417	335	280	241	211	188	169	154	141	131	121	113	106	100	94
9-#9	.0237	.250	437	350	291	250	218	194	175	159	146	134	125	117	109	103	97
10-#9	.0263	.256	457	364	302	258	226	200	180	164	150	138	128	120	112	106	100
11-#9	.0290	.262	477	378	313	267	233	207	186	168	154	142	132	123	115	108	102
12-#9	.0316	.267	497	392	324	276	240	213	191	173	158	146	135	126	118	111	105
6-#10	.0208	.241	409	330	276	237	208	186	167	152	140	129	120	112	105	99	94
7-#10	.0234	.249	435	348	290	249	218	194	174	158	145	134	125	116	109	103	97
8-#10	.0267	.257	460	366	304	260	227	201	181	164	150	139	129	120	113	106	100
9-#10	.0301	.264	486	384	318	271	236	210	188	170	156	144	134	124	117	110	104
10-#10	.0334	.270	511	402	332	382	246	217	195	177	162	149	138	129	120	113	107
11-#10	.0368	.277	536	420	344	292	254	225	201	182	167	153	142	132	124	116	110
6-#11	.0246	.252	444	355	295	253	221	196	177	161	147	136	126	118	110	104	98
7-#11	.0288	.261	475	376	312	266	232	206	185	168	154	142	132	123	115	108	102
8-#11	.0328	.270	507	399	329	280	244	216	194	175	160	148	137	128	120	112	106
9-#11	.0370	.278	538	421	346	293	255	225	202	183	167	154	142	133	124	117	110
10-#11	.0411	.285	569	443	362	307	266	235	210	190	173	160	148	138	129	121	114
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—24 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#7	.0133	.202	425	353	302	265	235	211	192	176	162	151	141	132	124	117	111
12-#7	.0159	.207	449	372	317	277	245	220	200	183	169	157	146	137	129	122	115
8-#8	.0140	.203	431	358	307	268	238	214	194	178	164	152	142	133	125	118	112
10-#8	.0175	.211	463	382	326	284	251	225	204	187	172	160	149	139	131	124	117
12-#8	.0210	.218	495	406	345	299	264	237	214	196	180	167	156	146	137	129	122
6-#9	.0133	.202	425	353	302	265	235	211	192	176	162	151	141	132	124	117	111
8-#9	.0177	.211	465	384	327	285	252	226	205	188	173	160	150	140	132	124	118
10-#9	.0221	.220	505	414	350	304	269	240	218	199	183	170	158	148	139	131	124
11-#9	.0243	.224	525	429	362	314	277	248	224	204	188	174	162	152	142	134	127
12-#9	.0265	.228	545	443	374	323	285	254	230	210	193	179	166	155	146	137	130
14-#9	.0310	.236	585	473	397	342	301	268	242	220	202	187	174	163	153	144	136
6-#10	.0169	.210	457	378	322	280	248	223	202	185	170	158	147	138	130	122	116
7-#10	.0197	.215	483	397	338	294	260	233	211	193	177	164	153	144	135	127	120
8-#10	.0225	.221	508	416	352	305	269	241	218	199	183	170	158	148	139	131	124
9-#10	.0253	.226	534	436	368	318	280	250	227	207	190	176	164	153	144	136	128
10-#10	.0281	.231	559	454	382	330	290	259	234	214	196	182	169	158	148	139	132
12-#10	.0337	.240	610	492	412	354	311	277	250	228	209	193	179	168	157	148	140
13-#10	.0365	.244	635	510	426	366	321	286	257	234	215	199	184	172	162	152	144
6-#11	.0207	.217	492	404	343	298	263	236	214	195	180	167	155	145	136	129	122
7-#11	.0242	.224	523	427	361	313	276	246	223	204	187	173	161	151	142	134	127
8-#11	.0276	.230	555	451	380	328	289	258	233	213	195	181	168	157	148	139	132
9-#11	.0311	.236	586	474	398	343	301	269	243	221	203	188	174	163	153	144	136
10-#11	.0345	.242	617	497	416	357	313	279	252	229	210	194	180	168	158	149	141
11-#11	.0379	.247	648	520	433	372	326	290	261	237	218	201	187	174	164	154	145
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—26 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#8	.0134	.187	500	421	364	320	286	258	236	216	200	186	174	164	154	146	138
11-#8	.0164	.193	532	446	384	337	300	271	247	226	209	194	182	170	160	152	144
13-#8	.0193	.198	563	470	403	353	314	283	257	236	218	202	189	177	167	157	149
15-#8	.0223	.204	595	494	422	369	328	294	267	245	226	210	196	183	172	163	154
7-#9	.0132	.186	498	420	363	320	285	258	236	216	200	186	174	164	154	146	138
8-#9	.0151	.190	518	435	375	330	294	266	242	222	206	191	178	168	158	149	141
9-#9	.0169	.194	538	450	387	340	303	273	249	228	211	196	183	172	162	153	145
10-#9	.0188	.197	558	466	400	350	312	281	256	235	217	201	188	176	166	157	149
11-#9	.0207	.200	578	481	412	361	321	289	263	241	222	206	193	180	170	160	152
12-#9	.0226	.204	598	496	424	371	329	296	269	246	227	211	197	184	173	164	155
13-#9	.0245	.207	618	512	437	381	338	304	276	252	233	216	201	189	177	167	158
15-#9	.0282	.214	658	542	461	401	355	318	288	263	243	225	210	196	184	174	165
6-#10	.0143	.188	510	429	371	326	291	263	240	220	204	190	177	166	157	148	140
8-#10	.0191	.198	561	468	402	352	313	282	256	235	217	201	188	176	166	157	149
10-#10	.0239	.206	612	507	433	378	335	301	274	250	231	214	200	187	176	166	158
12-#10	.0287	.214	663	546	464	404	357	320	290	266	245	227	211	198	186	175	166
14-#10	.0335	.221	714	585	495	429	379	339	307	280	258	239	222	208	196	184	174
6-#11	.0176	.195	545	456	392	344	306	276	251	230	213	198	185	173	163	154	146
7-#11	.0206	.200	576	480	411	360	320	288	262	240	221	206	192	180	169	160	152
8-#11	.0235	.206	608	504	430	376	333	300	272	249	229	213	199	186	175	165	156
9-#11	.0264	.211	639	527	449	391	346	311	282	258	238	220	205	192	181	171	162
10-#11	.0294	.216	670	551	468	406	359	322	292	267	245	227	212	198	187	176	166
11-#11	.0323	.220	701	574	487	422	373	334	302	276	254	235	219	205	192	182	172
13-#11	.0382	.228	764	622	525	453	399	357	322	294	270	250	233	218	205	193	182
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—28 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#8	.0128	.172	574	490	427	379	340	309	283	260	242	225	211	198	187	177	168
12-#8	.0154	.177	606	515	447	396	355	321	294	271	251	234	219	205	194	183	174
14-#8	.0179	.181	637	539	468	413	369	334	306	281	260	242	227	213	201	190	180
8-#9	.0130	.172	576	492	428	380	341	310	284	261	242	226	212	199	188	178	169
10-#9	.0162	.178	616	522	454	401	360	326	298	274	254	237	222	208	196	186	176
12-#9	.0195	.184	656	554	479	423	378	341	312	286	265	247	231	217	204	193	183
14-#9	.0227	.190	696	585	505	443	396	357	325	299	276	257	240	225	212	200	190
7-#10	.0144	.175	594	505	440	390	350	317	290	267	247	231	216	203	192	181	172
8-#10	.0165	.179	619	525	455	403	361	327	298	275	254	237	222	208	197	186	177
9-#10	.0186	.183	645	545	472	416	372	337	307	283	262	244	228	214	202	191	181
10-#10	.0206	.186	670	565	488	430	384	347	317	291	269	250	234	220	207	196	186
11-#10	.0227	.190	695	584	504	443	395	356	325	298	276	256	240	225	212	200	190
12-#10	.0248	.193	721	604	520	456	406	367	334	306	283	264	246	231	217	205	195
13-#10	.0268	.196	746	624	535	470	418	377	343	314	290	270	252	236	222	210	199
14-#10	.0289	.199	772	644	552	483	430	387	352	322	298	277	258	242	228	215	204
6-#11	.0152	.177	603	512	445	394	353	320	292	269	250	233	218	205	193	183	173
7-#11	.0177	.181	634	537	465	411	368	333	304	279	259	241	226	212	200	189	179
8-#11	.0203	.186	666	561	485	428	382	345	315	289	268	249	233	219	206	195	185
9-#11	.0228	.190	697	585	505	444	396	357	326	299	276	257	240	226	212	201	190
10-#11	.0253	.194	728	610	525	460	410	370	336	309	286	265	248	232	219	207	196
11-#11	.0278	.198	759	633	544	476	424	382	347	318	294	273	255	239	225	212	201
12-#11	.0304	.201	790	658	564	493	438	394	358	328	303	281	262	246	232	219	207
13-#11	.0330	.205	822	682	583	509	452	406	368	338	312	289	270	252	238	224	212
14-#11	.0355	.208	853	706	602	525	465	418	379	348	320	297	277	259	244	230	218
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—30 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#9	.0127	.158	657	569	502	449	405	370	341	315	294	274	258	243	230	218	207
11-#9	.0156	.162	697	600	526	469	423	385	353	327	304	284	266	250	237	224	213
13-#9	.0184	.166	737	632	554	493	443	403	370	340	316	296	278	261	246	234	222
15-#9	.0212	.170	777	663	579	513	461	419	383	354	328	306	287	270	254	241	229
8-#10	.0144	.160	680	585	515	458	414	377	346	320	297	278	261	246	232	220	209
9-#10	.0162	.163	706	608	534	476	430	392	360	332	309	288	271	255	241	229	217
10-#10	.0180	.166	731	628	550	490	441	402	368	340	316	294	276	260	246	233	221
11-#10	.0198	.168	756	650	566	503	452	411	378	348	322	302	283	266	251	238	226
12-#10	.0216	.171	782	669	584	518	465	423	387	357	332	309	290	272	257	244	231
13-#10	.0234	.173	807	688	600	532	476	433	396	366	338	316	296	278	263	248	236
14-#10	.0252	.175	833	710	618	546	490	445	406	374	347	324	303	285	269	254	242
15-#10	.0270	.178	858	730	634	560	501	454	415	382	354	330	309	291	274	259	246
16-#10	.0287	.180	883	746	648	572	512	463	423	389	360	336	314	295	278	264	250
17-#10	.0305	.182	909	767	665	587	526	475	434	400	370	345	322	302	285	270	256
6-#11	.0132	.158	664	574	505	450	406	370	340	315	292	274	256	242	228	216	206
7-#11	.0154	.162	695	598	525	468	423	384	352	326	304	284	265	250	236	224	213
8-#11	.0176	.165	727	624	546	486	438	398	366	338	314	292	274	258	244	231	220
9-#11	.0199	.168	758	650	568	505	453	412	378	349	323	302	283	266	252	238	226
10-#11	.0221	.172	789	675	588	522	470	426	391	360	334	312	292	275	260	246	234
11-#11	.0243	.174	820	698	609	538	484	438	401	370	343	319	300	282	266	251	239
12-#11	.0265	.177	851	725	630	558	500	453	415	382	354	329	308	290	274	259	246
13-#11	.0287	.180	883	750	651	575	515	466	427	393	364	339	317	298	281	266	252
14-#11	.0309	.182	914	772	670	591	529	478	437	402	372	347	324	304	287	271	258
15-#11	.0331	.185	945	798	690	609	545	492	449	413	382	356	333	312	294	279	264
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—32 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#9	.0149	.152	783	680	600	538	487	445	410	379	354	331	311	294	277	264	250
14-#9	.0174	.155	823	714	629	562	508	463	426	394	368	344	323	304	288	273	260
16-#9	.0199	.157	863	746	656	585	529	432	444	410	381	356	335	316	298	283	269
18-#9	.0224	.161	903	776	682	609	549	499	458	424	394	368	345	324	306	290	276
20-#9	.0249	.164	943	810	710	630	570	518	475	439	408	382	357	336	318	301	286
10-#10	.0158	.152	797	690	610	547	495	452	416	385	358	336	315	297	281	266	254
11-#10	.0174	.155	822	711	628	561	508	463	426	394	367	344	322	304	288	273	260
12-#10	.0190	.156	848	733	645	575	520	475	436	403	374	350	329	310	294	278	264
13-#10	.0205	.159	873	755	663	592	535	486	447	414	385	360	337	318	300	285	270
14-#10	.0221	.161	899	774	680	606	547	498	458	422	393	367	344	324	307	291	276
15-#10	.0237	.162	924	794	696	620	659	510	468	431	400	374	352	331	313	296	281
16-#10	.0253	.164	949	815	714	635	571	520	476	441	409	382	358	338	318	302	287
17-#10	.0268	.166	975	835	730	649	585	531	486	450	418	389	365	344	324	308	292
18-#10	.0284	.168	1000	855	748	665	596	542	497	458	425	396	372	350	330	313	297
8-#11	.0155	.152	793	688	608	545	494	450	415	384	358	335	315	297	281	267	254
9-#11	.0175	.154	824	714	629	562	509	464	427	395	368	344	323	304	288	274	260
10-#11	.0194	.157	855	740	650	580	524	478	440	406	378	353	332	313	296	280	266
11-#11	.0213	.160	886	765	671	600	540	493	453	419	389	364	341	321	304	288	274
12-#11	.0233	.162	917	788	691	616	555	505	464	428	398	372	348	328	310	294	279
13-#11	.0252	.164	949	815	714	635	571	520	476	440	409	382	358	337	318	302	287
14-#11	.0272	.167	980	840	735	652	588	535	490	452	420	392	368	346	326	309	294
15-#11	.0291	.169	1011	862	753	667	600	545	499	461	427	399	373	350	332	314	298
16-#11	.0310	.171	1042	892	780	690	622	565	518	476	443	410	386	362	342	325	308
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3000$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—34 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
15-#9	.0165	.144	914	798	709	636	578	530	489	454	423	396	373	352	334	316	302
17-#9	.0187	.147	954	832	736	661	601	550	506	470	439	410	386	365	345	328	312
19-#9	.0209	.150	994	865	765	685	621	568	524	485	452	423	398	375	356	337	321
10-#10	.0140	.141	868	760	676	610	555	510	470	437	408	382	360	340	323	307	292
11-#10	.0154	.143	893	781	696	626	570	522	483	448	419	393	370	348	330	314	299
12-#10	.0168	.145	919	804	712	640	581	533	491	456	426	398	375	354	335	318	304
13-#10	.0182	.147	944	824	730	657	595	545	503	466	435	408	384	362	343	326	310
14-#10	.0196	.148	970	845	749	670	608	556	512	475	443	414	390	368	348	331	314
15-#10	.0210	.150	995	865	766	686	622	569	524	486	453	424	398	376	356	337	321
16-#10	.0224	.152	1020	885	781	700	635	580	533	494	460	430	405	382	361	343	326
17-#10	.0238	.153	1051	910	805	720	650	594	546	506	471	440	414	390	369	350	334
18-#10	.0252	.155	1071	930	820	734	655	605	558	517	481	450	422	399	376	357	340
19-#10	.0266	.156	1097	950	835	748	675	616	566	525	488	455	428	405	382	362	334
8-#11	.0137	.140	864	755	674	610	552	507	468	435	405	381	359	339	322	305	290
9-#11	.0155	.143	895	781	695	625	568	520	480	446	416	390	367	346	328	312	297
10-#11	.0172	.145	926	810	720	646	586	537	495	460	429	402	378	358	338	321	306
11-#11	.0189	.148	957	834	736	660	598	547	505	467	435	408	383	362	342	325	309
12-#11	.0206	.149	988	860	760	682	617	565	520	482	450	420	395	373	354	335	319
13-#11	.0224	.152	1020	885	781	700	635	580	533	494	460	430	405	382	361	343	326
14-#11	.0241	.154	1051	911	805	723	653	596	550	509	473	443	416	392	372	352	335
15-#11	.0258	.156	1082	936	826	740	668	610	560	519	483	452	424	400	378	359	341
16-#11	.0275	.157	1113	964	846	755	685	624	574	531	494	462	434	408	386	366	349
17-#11	.0292	.159	1145	990	870	775	700	639	586	543	505	472	442	417	394	374	355
18-#11	.0310	.161	1176	1011	890	792	715	651	598	552	514	480	451	425	402	380	362
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $\qquad f_s = 20,000$ psi

COLUMN SIZE—14 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#5	.0121	.342	167	124	99	82	70	62	55	49	45	41	22	20	17	15	13
8-#5	.0161	.360	180	132	105	86	74	64	57	51	46	42	25	22	20	18	17
10-#5	.0201	.377	192	139	109	90	76	66	59	53	48	44	27	25	22	21	19
6-#6	.0171	.365	183	134	106	87	74	65	57	51	47	43	25	23	21	19	17
7-#6	.0200	.377	192	140	109	90	77	67	59	53	48	44	27	25	22	20	19
8-#6	.0228	.387	200	144	113	92	78	68	60	54	49	45	29	26	24	22	20
9-#6	.0257	.399	209	149	116	95	80	70	62	55	50	46	31	28	26	23	22
6-#7	.0234	.390	202	145	113	93	79	68	60	54	49	45	30	26	24	22	20
7-#7	.0273	.404	214	152	118	97	82	71	62	56	51	46	32	29	26	24	22
8-#7	.0312	.420	226	159	123	100	84	73	64	57	52	47	34	30	28	25	24
6-#8	.0308	.418	225	159	123	100	84	73	64	57	52	47	33	30	28	25	23
7-#8	.0359	.436	241	168	129	104	88	76	67	59	54	49	36	32	30	27	25
8-#8	.0410	.453	256	176	134	108	91	78	69	61	55	50	38	35	32	29	27
6-#9	.0390	.447	250	173	132	107	90	77	68	60	55	50	37	33	31	28	26
7-#9	.0454	.468	270	184	140	112	94	81	71	63	57	52	41	37	34	31	29
6-#10	.0495	.481	282	190	144	115	96	83	72	65	58	53	43	39	35	33	30
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	†	†	†	†	†

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities
$f'_c = 3750$ psi $\qquad f_s = 20,000$ psi

COLUMN SIZE—16 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#6	.0131	.301	223	171	139	117	101	89	80	72	65	60	56	33	30	26	24
9-#6	.0197	.323	249	188	151	126	109	95	85	76	69	64	59	39	36	33	30
6-#7	.0179	.317	242	184	148	124	107	94	83	75	68	63	58	38	34	31	29
7-#7	.0209	.327	254	192	154	128	110	97	86	78	70	64	59	41	37	34	31
9-#7	.0269	.345	278	207	164	137	117	102	90	81	74	68	62	46	41	38	35
11-#7	.0328	.362	302	222	175	145	123	108	95	85	78	71	65	50	46	42	39
6-#8	.0236	.336	265	198	158	132	113	99	88	79	72	66	61	43	39	36	33
7-#8	.0275	.348	281	208	166	137	118	103	91	82	74	68	63	46	42	39	36
8-#8	.0314	.359	296	218	172	143	122	106	94	84	76	70	64	49	45	41	38
9-#8	.0353	.370	312	228	179	148	126	109	97	87	79	72	66	52	48	44	41
6-#9	.0298	.354	290	214	170	140	120	105	93	83	76	69	64	48	44	40	37
7-#9	.0348	.368	310	226	178	147	125	109	96	87	78	72	66	52	47	43	40
8-#9	.0398	.381	330	239	187	154	131	113	100	90	82	74	68	55	51	47	43
6-#10	.0379	.377	322	234	184	151	128	112	99	88	80	73	68	54	49	45	42
7-#10	.0443	.392	348	250	195	160	135	118	104	93	84	77	71	59	54	50	46
6-#11	.0466	.398	357	255	199	163	138	119	105	94	85	78	72	60	55	51	47
7-#11	.0544	.417	388	274	212	172	145	126	111	99	90	82	75	66	61	56	52
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	†	†	†	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line and below zigzag horizontal line of each group, concrete governs and safe load, N, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—18 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#7	.0141	.271	287	226	187	158	138	122	109	99	90	83	77	72	67	42	39
9-#7	.0212	.291	323	250	204	172	149	132	118	106	97	89	83	77	72	51	47
11-#7	.0259	.305	347	266	216	181	156	137	123	111	101	93	86	80	74	56	52
6-#8	.0186	.284	310	241	198	167	145	128	114	104	95	87	81	75	70	48	44
7-#8	.0217	.293	326	252	206	174	150	132	118	107	97	90	83	77	72	52	48
8-#8	.0248	.302	341	262	212	179	154	136	121	110	100	92	85	79	74	55	51
9-#8	.0279	.310	357	272	220	185	159	140	125	113	103	94	87	81	76	58	54
11-#8	.0341	.326	389	293	236	196	169	148	132	118	108	99	91	85	79	65	60
6-#9	.0235	.298	335	258	210	177	153	134	120	109	99	91	84	78	73	54	50
7-#9	.0275	.309	355	271	219	184	159	139	124	112	102	94	87	81	75	58	53
8-#9	.0314	.319	375	284	229	192	165	144	129	116	106	97	89	83	78	62	57
9-#9	.0353	.329	395	297	238	199	171	149	133	120	109	100	92	86	80	66	61
10-#9	.0392	.338	415	310	248	206	176	154	137	123	112	103	95	88	82	70	65
6-#10	.0299	.315	367	279	225	189	162	143	127	115	104	96	88	82	77	60	56
7-#10	.0349	.327	393	296	237	198	170	149	133	120	109	100	92	86	80	66	61
8-#10	.0399	.339	418	312	249	208	178	155	138	124	113	103	95	88	82	71	66
9-#10	.0448	.350	444	329	261	217	185	162	143	129	117	107	99	92	85	76	70
6-#11	.0367	.332	402	302	241	201	173	151	134	121	110	101	93	86	81	67	62
7-#11	.0429	.345	433	322	256	213	182	159	141	126	115	105	97	90	84	74	69
8-#11	.0490	.359	465	342	270	224	191	166	147	132	120	110	101	94	88	80	74
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	†	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—20 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
6-#8	.0151	.243	360	289	242	208	183	162	146	133	122	113	105	98	92	86	58
7-#8	.0176	.249	376	301	251	215	188	168	151	137	126	116	108	101	94	89	62
8-#8	.0201	.256	391	312	258	221	193	171	154	140	128	118	110	102	96	90	67
9-#8	.0226	.262	407	322	267	228	199	177	159	144	132	121	113	105	98	93	70
10-#8	.0252	.267	423	334	276	235	204	181	163	147	135	124	115	107	100	95	74
11-#8	.0277	.272	439	345	284	242	210	186	167	151	138	127	118	110	103	97	78
12-#8	.0302	.278	455	356	292	248	215	190	170	154	141	130	120	112	105	98	81
6-#9	.0191	.253	385	307	256	219	191	170	153	139	127	118	109	102	95	90	65
7-#9	.0223	.260	405	321	266	227	198	176	158	144	131	121	112	105	98	92	70
8-#9	.0255	.268	425	335	277	236	205	182	163	148	135	125	115	108	101	95	74
9-#9	.0287	.275	445	349	287	244	212	187	168	152	139	128	119	111	103	97	79
10-#9	.0318	.281	465	363	297	259	219	193	173	157	143	132	122	114	106	100	84
11-#9	.0351	.288	485	376	308	260	225	199	178	161	147	135	125	116	109	102	88
6-#10	.0243	.265	417	330	273	232	202	179	161	146	134	123	114	106	100	94	73
7-#10	.0283	.274	443	347	286	243	211	187	167	152	139	128	118	110	103	97	79
8-#10	.0324	.282	468	365	299	253	220	194	174	157	144	132	122	114	107	100	84
9-#10	.0364	.298	494	383	313	265	230	202	181	163	149	137	127	118	111	104	90
10-#10	.0404	.297	519	400	325	274	237	209	186	169	154	141	131	122	114	107	95
6-#11	.0298	.277	452	354	291	247	214	189	170	154	141	129	120	112	104	98	81
7-#11	.0348	.287	483	375	307	260	225	198	177	161	147	135	125	116	109	102	88
8-#11	.0398	.296	515	397	323	273	236	208	185	168	153	140	130	121	113	106	95
9-#11	.0447	.304	546	419	340	286	247	217	193	174	159	146	135	126	118	110	101
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	†

Outside diameter of spiral should be 3 in. less than outside diameter of column.

† To right of vertical line concrete governs and safe load, *N*, is the same for 16,000 psi steel.

SPIRALLY REINFORCED ROUND COLUMNS—

Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $f_s = 20{,}000$ psi

COLUMN SIZE—22 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
7-#8	.0146	.220	432	354	300	260	230	206	186	170	156	145	135	126	119	112	106
8-#8	.0166	.225	447	365	308	267	235	210	190	173	160	148	138	129	121	114	108
9-#8	.0187	.229	463	377	317	274	242	216	195	178	163	151	141	132	124	116	110
10-#8	.0208	.233	479	388	327	282	248	221	200	182	167	155	144	134	126	119	112
11-#8	.0229	.238	495	400	335	289	254	226	204	186	170	158	146	137	128	121	114
12-#8	.0250	.242	511	411	344	296	260	231	209	190	174	161	150	140	131	123	116
6-#9	.0158	.223	441	360	305	264	233	208	189	172	158	147	137	128	120	113	107
7-#9	.0184	.229	461	375	316	273	240	215	194	177	163	150	140	131	123	116	109
8-#9	.0211	.234	481	390	327	283	248	222	200	183	168	155	144	134	126	126	112
9-#9	.0237	.240	501	404	339	292	256	228	206	188	172	159	148	138	130	122	115
10-#9	.0263	.245	521	418	349	300	263	234	211	182	176	163	151	141	132	124	118
11-#9	.0290	.250	541	433	360	309	270	240	216	197	180	166	154	144	135	127	120
12-#9	.0316	.255	561	447	372	318	278	247	222	202	185	170	158	147	138	130	123
6-#10	.0208	.232	473	384	323	279	245	219	198	180	166	153	142	133	125	118	111
7-#10	.0234	.239	499	402	338	290	255	227	205	187	171	158	147	137	129	121	115
8-#10	.0267	.245	524	420	352	302	265	236	212	193	177	163	152	142	133	125	118
9-#10	.0301	.252	550	439	365	313	274	243	219	199	182	168	156	146	137	129	121
10-#10	.0334	.258	575	457	379	324	283	251	225	205	188	173	160	150	140	132	125
11-#10	.0368	.264	600	474	392	334	292	259	232	211	193	178	165	154	144	135	128
6-#11	.0246	.241	508	409	343	295	259	230	208	189	174	160	149	139	131	123	116
7-#11	.0288	.250	539	431	359	308	269	239	216	196	180	166	154	144	135	127	120
8-#11	.0328	.257	571	454	377	322	281	250	224	204	187	172	160	149	140	132	124
9-#11	.0370	.265	602	476	394	335	292	259	233	211	193	178	165	154	144	135	128
10-#11	.0411	.271	633	495	410	348	303	268	240	218	199	184	170	159	148	140	132
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—24 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	$M/N = e$ (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#7	.0133	.196	502	420	360	316	281	254	231	212	195	182	170	159	150	141	134
12-#7	.0159	.201	526	438	375	328	291	262	238	218	202	187	175	164	154	146	138
8-#8	.0140	.198	508	424	364	318	283	255	232	213	196	182	170	160	151	142	135
10-#8	.0175	.204	540	448	383	335	297	267	242	222	205	190	178	166	156	148	140
12-#8	.0210	.210	572	472	403	351	311	279	253	232	213	198	185	173	162	153	145
6-#9	.0133	.196	502	420	360	316	281	254	231	212	195	182	170	159	150	141	134
8-#9	.0177	.204	542	450	385	336	299	268	244	223	206	191	178	167	157	149	141
10-#9	.0221	.212	582	480	408	356	315	282	256	234	216	200	186	175	164	155	147
11-#9	.0243	.216	602	495	420	365	323	289	262	240	220	204	190	178	168	158	150
12-#9	.0265	.220	622	510	432	375	331	296	268	245	225	209	194	182	171	161	152
14-#9	.0310	.226	662	540	456	394	348	311	281	256	236	218	203	190	178	168	159
6-#10	.0169	.202	534	444	380	332	295	266	242	221	204	189	177	166	156	147	139
7-#10	.0197	.208	560	463	395	344	306	274	249	228	210	195	182	170	160	151	143
8-#10	.0225	.213	585	482	410	357	316	283	257	235	216	200	187	175	164	155	147
9-#10	.0253	.218	611	501	426	369	326	292	265	242	223	206	192	180	169	159	151
10-#10	.0281	.222	636	520	440	382	337	301	273	249	229	212	198	185	174	164	155
12-#10	.0337	.230	687	559	471	407	358	320	289	263	242	224	208	195	183	172	163
13-#10	.0365	.235	712	576	484	418	367	327	295	269	247	229	213	199	186	176	166
6-#11	.0207	.210	569	470	400	349	309	277	252	230	212	197	184	172	162	153	144
7-#11	.0242	.216	600	494	419	364	322	288	261	239	220	204	190	178	167	158	149
8-#11	.0276	.221	632	518	438	380	335	300	272	248	228	212	197	184	173	163	154
9-#11	.0311	.227	663	540	456	394	347	310	280	256	235	218	203	190	178	168	159
10-#11	.0345	.232	694	564	474	409	360	321	290	264	243	225	209	195	183	173	164
11-#11	.0379	.236	725	586	492	424	373	332	300	273	251	232	216	202	189	178	168
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—26 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#8	.0134	.182	590	499	432	382	341	309	282	260	240	224	209	196	185	175	166
11-#8	.0164	.187	622	524	452	398	356	321	293	269	249	232	217	204	192	181	172
13-#8	.0193	.192	653	548	472	415	369	333	304	279	258	239	224	210	198	187	177
15-#8	.0223	.196	685	573	492	431	384	346	315	289	267	248	232	217	204	193	183
7-#9	.0132	.180	588	498	432	382	342	309	283	260	241	224	210	197	186	176	167
8-#9	.0151	.184	608	513	444	392	350	316	289	266	246	229	214	201	189	179	170
9-#9	.0169	.187	628	529	457	402	359	324	296	272	252	234	219	206	194	182	174
10-#9	.0188	.191	648	544	469	412	367	332	302	277	256	238	223	209	197	186	176
11-#9	.0207	.194	668	560	481	422	376	339	309	283	262	243	227	213	201	190	180
12-#9	.0226	.196	688	575	494	433	385	347	316	290	268	249	232	218	205	194	184
13-#9	.0245	.200	708	590	506	442	394	354	322	295	272	253	236	221	208	197	186
15-#9	.0282	.205	748	620	530	463	411	369	335	307	283	263	245	230	216	204	193
6-#10	.0143	.183	600	507	439	387	346	313	286	263	243	226	212	199	188	178	168
8-#10	.0191	.191	651	547	471	414	369	333	304	279	258	240	224	210	198	187	172
10-#10	.0239	.198	702	586	503	440	392	353	321	294	271	252	236	221	208	196	186
12-#10	.0287	.206	753	625	533	465	412	371	337	308	284	264	246	230	217	205	194
14-#10	.0335	.212	804	663	565	492	435	390	354	324	298	276	258	241	227	214	203
6-#11	.0176	.188	635	534	463	406	362	327	298	274	253	236	220	207	195	184	175
7-#11	.0206	.193	666	558	480	422	376	339	309	283	262	244	227	213	201	190	180
8-#11	.0235	.198	698	582	500	438	389	351	319	293	279	251	234	220	207	195	185
9-#11	.0264	.202	729	606	519	454	403	362	330	302	279	259	241	226	213	201	190
10-#11	.0294	.207	760	630	537	469	415	373	339	310	286	265	248	232	218	206	195
11-#11	.0323	.211	791	653	556	484	429	385	349	319	294	273	254	238	224	211	200
13-#11	.0382	.218	854	701	595	516	456	408	370	338	311	288	268	251	236	223	211
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—28 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
10-#8	.0128	.168	677	580	507	450	405	368	337	311	289	270	253	238	225	213	202
12-#8	.0154	.172	709	605	527	468	420	381	349	322	299	278	261	245	231	219	208
14-#8	.0179	.175	740	630	548	485	435	394	361	333	308	287	269	253	238	226	214
8-#9	.0130	.168	679	581	509	451	406	369	338	312	290	270	253	238	225	213	203
10-#9	.0162	.173	719	612	534	473	425	385	353	325	301	281	263	248	234	221	210
12-#9	.0195	.178	759	644	560	495	443	401	367	338	313	292	273	257	242	229	217
14-#9	.0227	.183	799	675	585	516	461	417	381	350	324	302	282	265	250	236	224
7-#10	.0144	.170	697	596	520	462	415	377	345	318	295	276	258	243	230	217	206
8-#10	.0165	.173	722	615	536	475	427	387	354	327	303	282	264	249	235	222	211
9-#10	.0186	.177	748	636	552	489	488	397	363	334	310	288	270	254	240	227	215
10-#10	.0206	.180	773	655	568	502	449	406	371	342	316	295	276	259	244	231	220
11-#10	.0227	.183	798	674	584	515	460	417	380	350	324	301	282	265	250	236	224
12-#10	.0248	.186	824	695	601	529	472	427	390	358	331	308	288	271	255	241	229
13-#10	.0268	.189	849	714	616	542	484	436	398	365	338	314	294	276	260	246	233
14-#10	.0289	.192	875	734	632	555	495	446	407	373	345	320	300	281	265	250	237
6-#11	.0152	.171	706	603	526	466	419	380	348	321	298	278	260	245	231	219	208
7-#11	.0177	.175	737	628	546	484	434	393	359	331	307	286	268	252	238	225	214
8-#11	.0203	.179	769	652	566	500	448	406	370	342	316	294	276	259	244	231	219
9-#11	.0228	.183	800	675	586	516	461	418	381	350	324	302	283	266	250	237	224
10-#11	.0253	.186	831	700	605	533	476	430	393	361	334	310	290	273	257	243	230
11-#11	.0278	.190	862	724	624	549	490	442	403	370	342	318	297	279	263	248	236
12-#11	.0304	.194	893	747	643	565	503	453	412	379	350	325	304	285	268	254	240
13-#11	.0330	.196	925	775	664	582	519	467	425	390	360	334	313	293	276	261	247
14-#11	.0355	.199	956	797	683	599	532	479	435	399	369	342	320	300	282	266	252
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad$ $f_s = 20{,}000$ psi

COLUMN SIZE—30 IN. DIAMETER

Bars	p	$\frac{CD}{t}$	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
9-#9	.0127	.154	776	673	595	533	481	440	405	375	349	327	307	289	274	260	247
11-#9	.0156	.158	816	705	620	554	500	456	420	388	360	337	317	298	282	268	254
13-#9	.0184	.162	856	738	648	578	521	475	436	402	374	350	328	310	292	277	264
15-#9	.0212	.165	896	770	675	600	540	491	451	417	387	361	338	319	301	285	271
8-#10	.0144	.156	799	691	609	545	491	449	413	382	356	332	312	294	278	264	251
9-#10	.0162	.159	825	714	628	560	507	462	425	394	366	342	321	302	286	271	258
10-#10	.0180	.161	850	731	643	573	517	471	433	400	372	347	326	307	290	275	262
11-#10	.0198	.163	875	751	660	586	529	481	441	408	379	354	332	312	295	280	266
12-#10	.0216	.166	901	775	678	604	544	495	454	420	388	363	340	320	302	287	272
13-#10	.0234	.168	926	795	695	616	555	504	462	426	394	370	346	325	308	291	276
14-#10	.0252	.170	952	814	710	630	566	515	471	435	403	376	352	332	313	296	282
15-#10	.0270	.172	977	833	726	645	580	525	481	443	412	384	359	338	318	302	286
16-#10	.0287	.174	1002	855	745	658	591	535	490	453	420	390	366	344	325	307	292
17-#10	.0305	.176	1028	875	760	674	603	547	501	461	426	397	372	351	330	313	296
6-#11	.0132	.154	783	679	600	535	485	443	407	377	351	328	308	291	275	261	248
7-#11	.0154	.158	814	704	619	552	498	455	418	386	359	336	316	298	281	267	254
8-#11	.0176	.161	846	730	641	571	515	470	432	399	372	347	326	307	290	274	261
9-#11	.0199	.163	877	754	660	587	530	482	442	408	380	354	332	313	296	280	266
10-#11	.0221	.166	908	780	681	607	545	496	455	420	390	365	342	321	304	288	273
11-#11	.0243	.169	939	805	702	624	560	511	466	430	400	372	349	329	310	294	279
12-#11	.0265	.171	970	828	723	645	576	523	478	442	410	383	358	336	318	301	286
13-#11	.0287	.174	1002	855	745	658	591	535	490	453	420	390	366	344	325	307	292
14-#11	.0309	.176	1033	880	764	675	605	548	500	462	428	399	373	350	331	313	298
15-#11	.0331	.179	1064	905	786	695	624	565	515	476	441	411	384	361	341	322	306
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—32 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.) 0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
12-#9	.0149	.147	918	811	710	636	580	530	488	453	423	395	372	352	332	316	301
14-#9	.0174	.150	958	834	737	660	600	548	505	467	435	408	384	362	342	325	309
16-#9	.0199	.153	998	855	765	685	619	565	520	482	448	420	394	372	352	334	317
18-#9	.0224	.156	1038	897	790	705	638	580	534	495	460	430	404	380	360	341	325
20-#9	.0249	.159	1078	930	819	730	659	600	551	510	475	444	416	392	370	351	334
10-#10	.0158	.148	932	813	720	645	585	536	494	458	426	400	376	354	336	318	304
11-#10	.0174	.150	957	831	735	660	598	546	504	467	435	407	383	361	342	324	309
12-#10	.0190	.152	983	852	754	675	610	557	513	475	442	415	389	366	347	329	313
13-#10	.0205	.154	1008	875	773	692	625	572	526	488	453	425	399	376	356	338	321
14-#10	.0221	.156	1034	895	790	706	639	583	536	495	462	433	405	382	362	344	326
15-#10	.0237	.157	1059	915	805	716	647	591	544	502	466	436	410	387	366	346	330
16-#10	.0253	.159	1084	934	821	733	660	602	553	512	475	445	417	393	372	352	334
17-#10	.0268	.161	1110	953	838	746	674	614	562	520	484	452	424	398	376	357	340
18-#10	.0284	.162	1135	975	856	764	686	625	575	530	492	460	432	406	385	365	346
8-#11	.0155	.148	928	810	716	643	583	534	492	456	425	398	374	353	335	317	302
9-#11	.0175	.150	959	834	738	661	600	548	505	468	436	408	384	362	333	325	310
10-#11	.0194	.152	990	858	758	680	614	560	516	478	445	417	392	369	349	332	317
11-#11	.0213	.155	1021	885	780	698	630	575	530	490	456	427	400	378	358	339	322
12-#11	.0233	.157	1052	910	800	715	645	588	540	500	465	435	408	386	364	345	328
13-#11	.0252	.159	1084	935	821	733	661	603	554	511	475	445	416	393	372	352	335
14-#11	.0272	.161	1115	960	841	750	676	615	565	523	486	454	426	400	378	358	341
15-#11	.0291	.163	1146	985	865	770	694	632	580	535	497	465	436	411	388	368	349
16-#11	.0310	.165	1177	1005	885	785	710	642	588	548	507	473	443	418	393	373	354
For f_s = 16,000 psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

SPIRALLY REINFORCED ROUND COLUMNS—
Safe Load in Kips for Various Eccentricities

$f'_c = 3750$ psi $\qquad f_s = 20{,}000$ psi

COLUMN SIZE—34 IN. DIAMETER

Bars	p	CD/t	M/N = e (in.)														
			0	1	2	3	4	5	6	7	8	9	10	11	12	13	14
15-#9	.0165	.140	1067	943	840	756	689	631	584	542	506	475	446	423	400	380	362
17-#9	.0187	.143	1107	968	860	775	704	645	596	552	515	484	455	430	408	388	368
19-#9	.0209	.145	1147	1000	886	798	724	663	611	566	529	496	466	440	416	396	376
10-#10	.0140	.137	1021	899	800	722	658	605	560	520	485	455	430	406	385	366	349
11-#10	.0154	.139	1046	920	820	740	672	617	570	530	495	465	438	414	392	372	355
12-#10	.0168	.141	1072	940	835	752	685	629	580	540	504	472	445	420	398	379	360
13-#10	.0182	.142	1097	960	855	770	700	641	591	550	514	482	454	428	405	385	367
14-#10	.0196	.144	1123	980	870	784	710	651	600	558	520	487	459	434	410	389	371
15-#10	.0210	.145	1148	915	810	730	661	605	659	517	484	454	426	402	381	362	344
16-#10	.0224	.147	1173	1022	905	813	738	675	622	577	537	504	473	447	424	402	382
17-#10	.0238	.148	1204	1050	930	833	755	690	635	589	550	514	484	457	432	411	390
18-#10	.0252	.150	1224	1066	944	846	766	701	645	598	558	522	490	463	438	415	396
19-#10	.0266	.151	1250	1083	958	859	768	710	653	605	565	528	496	468	442	420	400
8-#11	.0137	.137	1017	895	798	720	655	602	557	518	483	454	427	405	384	365	347
9-#11	.0155	.139	1048	920	820	740	674	618	571	530	496	466	439	415	393	364	356
10-#11	.0172	.141	1079	945	840	758	690	632	585	544	507	475	448	423	401	381	363
11-#11	.0189	.143	1110	970	864	775	705	645	595	555	516	485	455	430	407	387	368
12-#11	.0206	.145	1141	996	885	795	723	661	610	567	529	495	466	440	416	395	377
13-#11	.0224	.147	1173	1022	906	815	740	676	624	578	540	505	475	450	425	404	384
14-#11	.0241	.148	1204	1050	926	830	754	689	635	588	548	512	482	455	430	409	389
15-#11	.0258	.151	1235	1071	948	850	770	705	648	602	560	525	492	464	440	416	397
16-#11	.0275	.152	1266	1098	970	868	785	716	660	611	569	534	500	471	446	423	403
17-#11	.0292	.154	1298	1123	994	888	804	734	675	625	581	544	511	483	455	432	412
18-#11	.0310	.155	1329	1150	1011	905	818	746	686	635	590	552	519	490	463	439	417
For $f_s = 16{,}000$ psi multiply by				.935	.945	.960	.960	.965	.965	.975	.980	.980	.980	.980	.980	.980	.980

Outside diameter of spiral should be 3 in. less than outside diameter of column.

$B = CD$ VALUES FOR $f_s = 16{,}000$ psi

In each of the three preceding sections, values of eccentrically applied loads on concrete columns are tabulated for a steel tension of $f_s = 20{,}000$ psi and factors are given to reduce these values for $f_s = 16{,}000$ psi. While using these factors is sufficiently accurate for practical purposes, it is possible to come somewhat closer by using the $B = CD$ values given in the following tables.

Example—In Ex. II on page 278, the actual and allowable stresses were computed for a 20 in. square tied concrete column with 6-#8 vertical bars and an eccentricity of 3 in. when $f'_c = 3000$ psi, $f_s = 0.8 \times 16{,}000 = 12{,}800$ psi and $n = 10$. Check this from the table on page 358.

$p_g = \dfrac{6 \times 0.79}{20 \times 20} = 0.01185$ and $g = \dfrac{11}{16} = 0.688$. From the table on page 358, $B = CD =$ 2.64, so $\dfrac{B}{t} = \dfrac{CD}{t} = 0.132$.

$$P = \begin{cases} \text{Concrete} \quad 20 \times 20 \text{ @ } 540 = 216{,}000 \text{ lb} \\ \text{Bars} \quad 6 \times 0.79 \text{ @ } 12{,}800 = \underline{60{,}600 \text{ lb}} \end{cases}$$
$$276{,}600 \text{ lb}$$

$$N = \frac{P}{1 + \dfrac{CDe}{t}} = \frac{P}{1 + \dfrac{Be}{t}} = \frac{276{,}600}{1 + 0.132 \times 3} = 198 \text{ kips}$$

According to Ex. II, page 278, the eccentric load, N, could be increased approximately in the ratio $\dfrac{1.000}{0.973} \times 193 = 198$ kips. This checks and indicates that the factor in this case is 2¾ per cent on the safe side.

VALUES OF $B = CD$
SQUARE COLUMNS WITH TIES
$f_s = 0.8 \times 16{,}000 = 12{,}800$ psi

p_g	CD (g = .65)		CD (g = .70)		CD (g = .75)		CD (g = .80)		CD (g = .85)	
	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750
.010	2.65	2.60	2.58	2.55	2.54	2.51	2.49	2.46	2.45	2.46
.011	2.68	2.62	2.58	2.58	2.55	2.54	2.51	2.49	2.45	2.47
.012	2.73	2.64	2.63	2.59	2.58	2.55	2.53	2.49	2.47	2.47
.013	2.75	2.67	2.65	2.60	2.60	2.55	2.55	2.52	2.50	2.47
.014	2.77	2.68	2.67	2.60	2.63	2.55	2.58	2.50	2.50	2.47
.015	2.80	2.72	2.69	2.62	2.64	2.57	2.59	2.53	2.52	2.47
.016	2.82	2.73	2.71	2.64	2.67	2.59	2.62	2.53	2.52	2.46
.017	2.82	2.74	2.73	2.65	2.68	2.60	2.63	2.55	2.53	2.48
.018	2.83	2.76	2.76	2.67	2.70	2.62	2.63	2.57	2.53	2.50
.019	2.86	2.77	2.78	2.67	2.70	2.64	2.63	2.59	2.53	2.50
.020	2.87	2.80	2.80	2.68	2.72	2.64	2.65	2.59	2.55	2.52
.021	2.88	2.81	2.83	2.71	2.73	2.67	2.65	2.62	2.55	2.52
.022	2.89	2.82	2.84	2.73	2.74	2.68	2.65	2.63	2.57	2.53
.023	2.92	2.84	2.87	2.75	2.77	2.70	2.67	2.64	2.57	2.53
.024	2.94	2.86	2.88	2.78	2.78	2.71	2.67	2.64	2.55	2.54
.025	2.97	2.87	2.88	2.80	2.77	2.72	2.68	2.65	2.57	2.55
.026	3.00	2.90	2.91	2.82	2.80	2.75	2.70	2.67	2.60	2.55
.027	3.01	2.91	2.93	2.84	2.82	2.74	2.68	2.66	2.58	2.57
.028	3.03	2.93	2.93	2.87	2.82	2.77	2.72	2.67	2.62	2.57
.029	3.04	2.94	2.93	2.88	2.83	2.78	2.70	2.68	2.60	2.58
.030	3.05	2.96	2.94	2.91	2.84	2.81	2.70	2.70	2.60	2.60
.031	3.07	2.97	2.96	2.92	2.85	2.83	2.72	2.70	2.62	2.58
.032	3.07	2.99	2.94	2.93	2.84	2.81	2.72	2.70	2.60	2.60
.033	3.08	3.01	2.96	2.94	2.85	2.83	2.69	2.72	2.58	2.62
.034	3.10	3.03	2.98	2.93	2.87	2.85	2.72	2.72	2.60	2.62
.035	3.10	3.05	2.98	2.94	2.85	2.83	2.68	2.73	2.58	2.63
.036	3.12	3.06	2.98	2.96	2.87	2.84	2.70	2.73	2.60	2.63
.037	3.14	3.07	3.00	2.98	2.90	2.87	2.72	2.74	2.62	2.63
.038	3.15	3.08	2.98	2.97	2.88	2.87	2.72	2.76	2.60	2.64
.039	3.17	3.09	2.99	2.98	2.88	2.87	2.72	2.76	2.60	2.63
.040	3.20	3.11	3.02	3.00	2.91	2.89	2.74	2.76	2.62	2.64

VALUES OF $B = CD$
SQUARE COLUMNS WITH SPIRALS
f_s = 16,000 psi

p_g	CD (g = .65)		CD (g = .70)		CD (g = .75)		CD (g = .80)		CD (g = .85)	
	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750	f'_c = 3000	f'_c = 3750
.010	3.52	3.42	3.52	3.44	3.46	3.40	3.42	3.35	3.38	3.34
.011	3.57	3.47	3.56	3.47	3.50	3.42	3.45	3.38	3.40	3.37
.012	3.60	3.51	3.60	3.50	3.54	3.45	3.48	3.42	3.42	3.38
.013	3.67	3.52	3.66	3.52	3.60	3.47	3.54	3.43	3.47	3.39
.014	3.72	3.56	3.69	3.53	3.62	3.50	3.57	3.45	3.50	3.41
.015	3.75	3.60	3.72	3.60	3.65	3.54	3.60	3.47	3.54	3.44
.016	3.81	3.65	3.75	3.63	3.68	3.57	3.62	3.51	3.56	3.45
.017	3.85	3.68	3.78	3.66	3.72	3.60	3.66	3.54	3.60	3.48
.018	3.86	3.75	3.80	3.72	3.75	3.66	3.67	3.60	3.61	3.54
.019	3.90	3.78	3.84	3.75	3.78	3.70	3.68	3.63	3.63	3.57
.020	3.95	3.83	3.87	3.78	3.82	3.72	3.72	3.66	3.65	3.60
.021	3.97	3.87	3.92	3.82	3.87	3.75	3.75	3.69	3.68	3.63
.022	4.06	3.92	4.00	3.85	3.93	3.78	3.80	3.72	3.75	3.66
.023	4.10	3.93	4.03	3.86	3.97	3.80	3.83	3.72	3.77	3.67
.024	4.14	3.95	4.06	3.90	4.00	3.84	3.87	3.75	3.80	3.68
.025	4.18	4.00	4.10	3.94	4.03	3.88	3.90	3.78	3.83	3.72
.026	4.23	4.04	4.12	3.97	4.06	3.91	3.92	3.82	3.86	3.75
.027	4.26	4.08	4.14	4.02	4.10	3.95	3.95	3.84	3.90	3.77
.028	4.28	4.09	4.15	4.04	4.11	3.97	3.97	3.85	3.91	3.78
.029	4.32	4.12	4.19	4.06	4.13	4.00	4.00	3.86	3.92	3.80
.030	4.35	4.16	4.25	4.10	4.15	4.03	4.05	3.90	3.95	3.84
.031	4.39	4.22	4.28	4.13	4.19	4.06	4.10	3.92	3.98	3.86
.032	4.41	4.25	4.35	4.16	4.22	4.10	4.13	3.95	4.00	3.90
.033	4.45	4.27	4.40	4.18	4.25	4.11	4.18	3.96	4.03	3.91
.034	4.49	4.30	4.42	4.20	4.28	4.13	4.20	3.97	4.06	3.92
.035	4.54	4.34	4.45	4.22	4.33	4.16	4.24	4.01	4.10	3.95
.036	4.59	4.39	4.48	4.25	4.36	4.19	4.27	4.05	4.12	3.98
.037	4.62	4.42	4.51	4.30	4.41	4.22	4.30	4.10	4.15	4.02
.038	4.66	4.42	4.55	4.31	4.47	4.23	4.33	4.11	4.18	4.03
.039	4.71	4.45	4.58	4.35	4.50	4.25	4.35	4.15	4.20	4.04
.040	4.76	4.48	4.62	4.40	4.53	4.28	4.40	4.18	4.24	4.07
.041	4.78	4.49	4.67	4.41	4.55	4.30	4.41	4.19	4.25	4.08
.042	4.83	4.50	4.68	4.45	4.57	4.32	4.42	4.22	4.27	4.10
.043	4.86	4.55	4.72	4.48	4.60	4.35	4.45	4.27	4.30	4.13
.044	4.90	4.57	4.75	4.49	4.62	4.37	4.46	4.28	4.31	4.14
.045	4.92	4.60	4.77	4.50	4.64	4.39	4.48	4.30	4.32	4.15
.046	4.96	4.64	4.80	4.55	4.66	4.43	4.50	4.32	4.34	4.18
.047	5.00	4.66	4.81	4.56	4.68	4.45	4.52	4.33	4.34	4.18
.048	5.02	4.68	4.84	4.57	4.70	4.47	4.55	4.36	4.34	4.21
.049	5.05	4.70	4.85	4.58	4.72	4.50	4.56	4.37	4.34	4.22
.050	5.10	4.75	4.87	4.60	4.74	4.54	4.57	4.40	4.35	4.25
.051	5.12	4.78	4.93	4.64	4.76	4.57	4.60	4.43	4.37	4.27
.052	5.18	4.80	4.97	4.65	4.78	4.58	4.64	4.43	4.40	4.28
.053	5.19	4.84	4.99	4.71	4.80	4.60	4.65	4.45	4.41	4.30
.054	5.20	4.86	5.02	4.73	4.83	4.61	4.67	4.46	4.43	4.31
.055	5.22	4.90	5.08	4.75	4.85	4.63	4.70	4.48	4.45	4.33
.056	5.28	4.94	5.11	4.79	4.87	4.66	4.73	4.52	4.50	4.36
.057	5.32	4.96	5.15	4.80	4.94	4.68	4.76	4.53	4.52	4.37
.058	5.34	5.00	5.16	4.85	4.95	4.70	4.77	4.54	4.53	4.38
.059	5.38	5.03	5.18	4.87	5.00	4.72	4.80	4.57	4.55	4.40

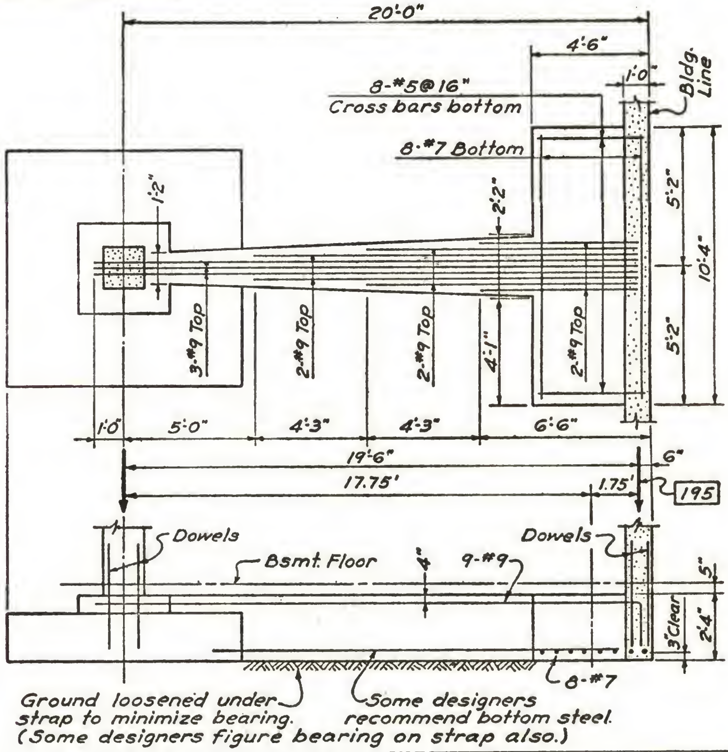
the backgrounds of show windows; (3) using a "strap" or "cantilevered" footing, where the bearing block is made large enough to carry all of the fulcrum loads, and a strap or cantilevered beam is provided back to an opposite interior column to balance the exterior column load across the bearing block; * and (4) a combined footing where the bearing block is made large enough so that one or more exterior columns and one or more interior columns rest upon the same block, which then becomes a partial mat with the same requirements regarding capacity for down-loads, and coincidence of upward and down-loads.

Combined footings can differ so much in the ratio of loads on interior and exterior columns, the distance between column centers, and the like that it is practically impossible to tabulate a sufficient range of choices. Therefore, illustrative problems for both a strap footing and a combined one are presented.

A word of caution—no tables or methods of computation can replace judgment, especially in dealing with soil conditions, where a clay, for example, may look as hard and firm as could be desired during one season, and at another time may be very questionable as a foundation material.

1. DESIGN OF A STRAP FOOTING.

Example—Given an exterior wall carrying 195 kips, whose exterior face is 20′-0″ from the center of an interior column, making the distance center to center 19′-6″. Using a soil pressure of not over 5000 psf, concrete strength of 3000 psi and steel at 20,000 psi, design a strap footing.



* A variation of this is to run a strap back into the building and, if there is no interior column readily available, provide sufficient dead weight of concrete to act as a balance for the eccentricity.

FOUNDATIONS AND FOOTINGS

Solution:—(See figure on page 362.)

Size of Bearing Block:— If the bearing block be assumed 4′-6″ wide, the distance from center of block to center of exterior column is 1.75 ft and to center of interior column is 17.75 ft, so, taking moments about the center of the bearing block, the uplift at the interior column (a downward load on the strap) is $\frac{1.75 \times 195{,}000}{17.75} = 19{,}200$ lb.

Column load	=	195,000 lb
Uplift reaction	=	19,200 lb
		214,200 lb
Weight of footing, assumed	=	18,000 lb
(Estimate as 8 to 9% of total load)		232,200 lb

Area required $= \frac{232{,}000}{5{,}000} = 46.4$ sf. Use 4′-6″ × 10′-4″ = 46.5 sf.

Net active bearing $= \frac{214{,}200}{4.5 \times 10.33} = 4610$ psf.

Assume projection beyond edge of strap as 4′-1″ and design as a cantilever, using a strip 1′-0″ wide:—

Shear at distance, d, from edge of strap (i.e. 4′-1″ − 2′-0″ = 2′-1″ from end of block) $= V = 4610 \times 2.08 = 9600$ lb

$M = 4610 \times 2.08^2 \times \frac{12}{2} = 461{,}000$ lb-in.

Try 28 in. depth, $d = 24$ in.,

$v = V/bjd = \frac{9600}{12 \times \frac{7}{8} \times 24} = 38 \text{ psi} < 75$

Block might be sloped or stepped down towards outer end but may be more economical carried straight through.

$R = M/bd^2 = \frac{461{,}000}{12 \times 24 \times 24} = 67 \text{ psi} < 235.$

For bars in bearing block see page 364.

Size of Strap:—Neglecting weight of strap,

Zero shear (max. moment) is $\frac{195{,}000}{10.33 \times 4610} = 4.10$ ft. from bldg line.

$M = 195{,}000 \left(\frac{4.10}{2} - 0.50\right) 12 = 3{,}625{,}000$ lb-in.

$V = 19{,}200$ lb

Try 28 in. depth, $d = 24$ in.,

$b_m = M/Rd^2 = \frac{3{,}625{,}000}{235 \times 24 \times 24} = 26.8$ in. at 4.10 ft from bldg line.

$b_v = V/vjd = \frac{19{,}200}{75 \times \frac{7}{8} \times 24} = 12.2$ in. throughout. Use 14 in.

Make strap 28 in. deep by 14 in. wide on edge of interior footing, about 26.8 in. wide 4.10 ft from bldg line, and 26 in. wide at edge of exterior footing.

Steel in Strap:—$A_s = M/f_s jd = \frac{3{,}625{,}000}{17{,}500 \times 24} = 8.62$ sq in.

Use 9-#9 = 9.00 sq in.

Number of top bars left for bond at interior column =

$N = V/jdou = \frac{19{,}200}{\frac{7}{8} \times 24 \times 3.5 \times 210} = 1\frac{1}{4}$ (Extend 3 and cut 6 short as below)

2—2/9 × 19.5 + 1′-5″ = 6′-0″
2—4/9 × 19.5 + 1′-5″ = 10′-3″
2—6/9 × 19.5 + 1′-5″ = 14′-6″
3—Full Length

FOUNDATIONS AND FOOTINGS

Steel in Bearing Block:— $A_s = \dfrac{M}{f_s jd} = \dfrac{442{,}500 \times 4.5}{17{,}500 \times 24} = 4.74$ sq in.

$8\text{-}\#7 = 4.80$ sq in.

$$u = \frac{vb}{\Sigma o} = \frac{73 \times 54}{8 \times 2.79} = 180 < 300 \text{ psi}$$

At distance d, beyond face of strap (i.e., at $5'\text{-}2'' - 1'\text{-}1'' - 2'\text{-}0'' = 2'\text{-}1''$ from end of bearing block):—

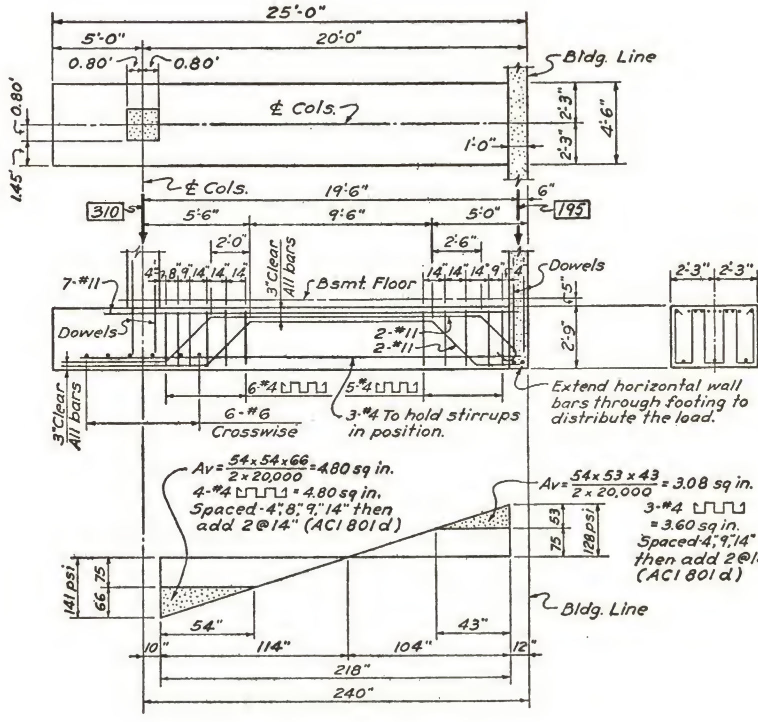
$$v_x = \frac{4610 \times 2.08}{12 \times 7/8 \times 24} = 38 < 75 \text{ psi, so no stirrups are required.}$$

Cross bars are proportioned arbitrarily as 8-#5 @ 16 in.

Check Weight:—$10.33 \times 4.5 \times 2.33$ @ $150 = 16{,}300$ lb (not including strap) < 18,000 lb, assumed.

2. DESIGN OF A COMBINED FOOTING.

Example—Given an exterior wall carrying 195 kips as in the preceding problem, whose exterior face is 20′-0″ from the center of an interior column carrying 310 kips and 1.6 ft square, making the distance center to center 19′-6″. Using a soil pressure of not over 5000 psf, concrete strength of 3000 psi and steel at 20,000 psi, design a combined footing.



Solution:—With columns 19.5 ft on centers and loads of 195 and 310 kips, the centroid of down-loads is $\left(0.5 + \dfrac{310}{505}19.5\right) = 12.47$ ft from the exterior face of the exterior column, so a rectangular footing must be 24.94 ft, say 25′-0″ long to center on the same line.

FOUNDATIONS AND FOOTINGS

The width is computed to furnish the required area:—

Load on exterior column = 195,000 lb
Load on interior column = 310,000 lb } 505,000 lb
Weight of footing, assumed
(Estimate 9 to 10% of total load) = 48,000 lb
553,000 lb

$$b = \frac{553{,}000}{5000 \times 25.0} = 4.43 \text{ ft, say } 4'\text{-}6''.$$

$$\text{Net soil pressure} = \frac{505{,}000}{4.5 \times 25.0} = 4490 \text{ psf.}$$

From exterior face to zero shear is computed:—

$$x = \frac{195{,}000}{4.5 \times 4490} = 9.65 \text{ ft.}$$

$$\text{Maximum bending moment, } M = 195{,}000 \times \left(\frac{9.65}{2} - 0.50\right) \times 12 = 10{,}120{,}000 \text{ lb-in.}$$

$$d = \sqrt{\frac{M}{Rb}} = \sqrt{\frac{10{,}120{,}000}{235 \times 54}} = 28.2, \text{ use 29 in.}; t = 2'\text{-}9''.$$

Because of tension in the top of the footing maximum shear is computed at the face of the interior column, (5.00 + 0.80 from inner end of footing):—

$V = 310{,}000 - 5.80 \times 4.5 \times 4490 = 193{,}000$ lb

$$v = V/bjd = \frac{193{,}000}{54 \times \frac{7}{8} \times 29} = 141 \text{ psi, so stirrups are required.}$$

Shear at inside face of exterior column $= 195{,}000 - 4.50 \times 1 \times 4490 = 174{,}800$ lb

$$v = V/bjd = \frac{174{,}800}{54 \times \frac{7}{8} \times 29} = 128 \text{ psi (stirrups)}$$

Computation of stirrups is shown in the figure on page 364. (See also pages 86-91.)

$$\text{Top steel:—}A_s = M/f_sjd = \frac{10{,}120{,}000}{17{,}500 \times 29} = 20.0 \text{ sq in.}$$

13-#11 = 20.3 sq in.

$$\text{Number of straight top bars:—}N = \frac{V}{uojd} = \frac{193{,}000}{210 \times 4.43 \times \frac{7}{8} \times 29} = 8.2 \text{ straight.}$$

Use 9 straight, bend 4.

$$\text{Bottom bars in cantilever:—}M = \frac{4.50 \times 4490 \times \overline{4.20}^2 \times 12}{2} = 2{,}140{,}000 \text{ lb-in.}$$

$$A_s = \frac{M}{f_sjd} = \frac{2{,}140{,}000}{17{,}500 \times 29} = 4.22 \text{ sq in.}$$

3-#11 = 4.68 sq in. (4 bent)

Since the point of inflection is only about 1.3 ft to the right of the interior column ($310 \times 1.3 = 403$; $4.5 \times 4490 \times 6.3 \times 3.15 = 401$) bottom bars in addition to truss bars and stirrup ties are not needed.

Bending bars:—(moment curve assumed parabolic and symmetrical about vertex at zero shear point)

Outside of zero shear:—

Bend 2 at $X_1 = 9.65\sqrt{2/13} = 3.80$ ft (5′-9″ from bldg line)

Bend 2 at $X_2 = 9.65\sqrt{4/13} = 5.38$ ft (1′-6″ outside of above)

Inside of zero shear:—

Bend 2 at $X_3 = 10.35\sqrt{2/13} = 4.04$ ft (4′-0″ from ℄ interior col.)

Bend 2 at $X_4 = 10.35\sqrt{4/13} = 5.71$ ft (1′-8″ outside of above)

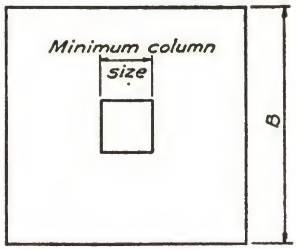
Cross bending at interior column:—[design as a cantilever for $\frac{310{,}000}{2}$ lb with arm = (1′-1½″ − 5″) = 8½″; $M = 1{,}320{,}000$ lb-in.]

$$A_s = \frac{M}{f_sjd} = \frac{1{,}320{,}000}{17{,}500 \times 29} = 2.60 \text{ sq in.}$$

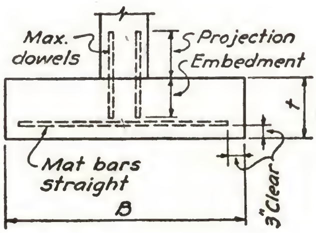
Use 6-#6 crosswise = 2.64 sq in.

Check assumed weight:—$4.50 \times 25.0 \times 2.75 \times 150 = 46{,}400$ lb

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS



f_s = 20,000 psi
f'_c = 3000 psi
f_c = 1350 psi
v_c = 75 psi
u = 240 psi



These tables give the safe total superimposed load (with dead weight of the footing deducted) that can be carried by reinforced concrete square individual column footings for soil capacities of 1,000, 2,000, 3,000, 4,000, 5,000, 6,000, 8,000 and 10,000 psf. The 1956 ACI "Building Code Requirements for Reinforced Concrete" has been followed.

Shear and bond stresses are based upon reinforcing bars with deformations meeting the requirements of ASTM A305. With this type of bar, hooking of the mat reinforcement is unnecessary and straight bars are used. Plain round bars or bars with deformations not meeting A305 cannot be used with these tables.

Designs are based upon a uniform depth, since the extra expense of sloped or stepped footings more than offsets the cost of added concrete to obtain the simple prism.

One grade of concrete is tabulated (f'_c = 3000 psi). If weaker concrete than this is used, the depths of footings must be increased to suit.

The columns in the tables headed "Maximum Size of Dowels" give the largest dowel which can be developed between the top of the footing and a point 3 in. above the subgrade for four conditions of allowable bond stress:—

Case 1: when the vertical column bars are of intermediate grade, deformed bars in tied columns, stressed 12,800 psi.

Case 2: intermediate grade bars in spirally reinforced columns or hard grade bars in tied columns, stressed 16,000 psi.

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

Case 3: hard grade bars in spirally reinforced concrete columns, stressed 20,000 psi.

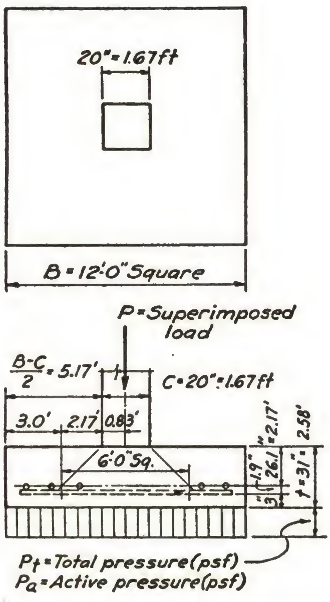
Case 4: using dowels embedded 20 diameters in the footing.*

Where footing is too shallow to develop dowels, use a larger number of smaller bars to increase bond or thicken footing either throughout or with a cap.

Not all possible designs are presented here. For those who wish to design beyond the scope of these tables, or merely see how they were prepared, the following example is presented:—

Example:—For the table on page 371 check the capacity of a 12 ft square footing on 5000 psf soil.

Tables are worked in reverse of an actual design; i.e., by determining the net capacity of a selected footing.



Solution— P = 664.3 kips (from table on page 371)

Footing Wt $= \dfrac{12 \times 12 \times 2.58 \text{ @ } 150}{1000} = \dfrac{55.7 \text{ kips}}{720.0 \text{ kips}}$ = 0.084 P (for estimating weight)

* The 1956 ACI Building Code requires that column verticals be lapped 20 diameters at the floor line, but this is partly to transmit possible bending moments, partly to get adequate practical lap and partly to allow for any possible variations in thickness of floor finishes. Many designers feel that it is impracticable to transmit moment into the footings and that the dowels need only transfer the vertical load in the bars at allowable bond stress.

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

$$\text{Total Soil Pressure, } p_t = \frac{720.0 \times 1000}{12 \times 12} = 5{,}000 \text{ psf}$$

$$\text{Net Soil Pressure, } p_a = \frac{664.3 \times 1000}{12 \times 12} = 4{,}610 \text{ psf}$$

Shear on a Section 6′-0″ Square: *—

$$v = \frac{V}{bdj} = \frac{(12 \times 12 - 6 \times 6)4610}{4 \times 6 \times 12 \times 26.1 \times \frac{7}{8}} = 75.7 \text{ psi vs } 75 \text{ psi allowed}$$

Moment on a Section at the Face of the Column, using 0.85 times the cantilever moment (ACI 1204 e):—

$$M = 0.85 \times 4610 \times (12 \times 5.17) \times 31 = 7{,}540{,}000 \text{ lb-in.}$$

$$R = \frac{M}{bd^2} = \frac{7{,}540{,}000}{144 \times 26.1 \times 26.1} = 76.8 < 235 \text{ so } \begin{cases} f_c < 1350 \text{ psi (table on p. 34)} \\ j = 0.916 \end{cases}$$

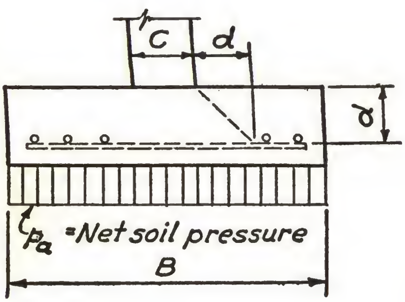
$$A_s = \frac{M}{f_s jd} = \frac{7{,}540{,}000}{20{,}000 \times 0.916 \times 26.1} = 15.77 \text{ sq in.}$$

13-#10 Bars = 16.5 sq in.

Bond on Bars at the Face of the Column:—

$$u = \frac{V}{\Sigma ojd} = \frac{4610 \times 12 \times 5.17}{13 \times 3.99 \times 0.916 \times 26.1} = 230 < 240 \text{ psi allowed}$$

* It is possible to determine the depth required for shear in a fairly simple manner.



$$d = \frac{[B^2 - (c + 2d)^2]p_a}{12 \times 4 \times v_c j(c + 2d)}$$

if $C = \dfrac{p_a}{504 v_c}$ and $k = \dfrac{C}{B}$, then:

$$\frac{d}{B} = \frac{\sqrt{2C + 4C^2 + \frac{1}{4}k^2} - \frac{1}{2}k(1 + 4C)}{2 + 4C}$$

For description of soil load test procedure, see page 274.

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

For explanation and limitations of these tables, see pages 366 to 368.

SOIL PRESSURE—1000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way			Max. Size of Dowels (see explanation on page 366)				Capacity (kips)
			Quant. of Bars	Bar Size	Spacing c/c (in.)	Case 1	Case 2	Case 3	Case 4	
5′-0	10½	10	5	#4	13	#5				21.7
5′-6	10½	10	6	#4	11½	#5				26.3
6′-0	10½	10	5	#5	16	#5				31.3
6′-6	10½	10	7	#5	11½	#5				36.7
7′-0	11	10	6	#6	15	#6				42.3
7′-6	12	10	6	#6	16	#6	#5			47.8
8′-0	12	10	8	#6	12½	#6	#5			54.4
8′-6	13	10	8	#6	13½	#7	#6			60.5
9′-0	13	10	10	#6	11	#7	#6			67.9
9′-6	14	10	10	#6	11½	#8	#6	#5		74.4
10′-0	15	10	8	#7	16	#9	#7	#5		81.3
10′-6	15	10	9	#7	14¾	#9	#7	#5		89.6
11′-0	16	10	10	#7	13½	#9	#7	#5	#5	96.8

SOIL PRESSURE—2000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way			Max. Size of Dowels (see explanation on page 366)				Capacity (kips)
			Quant. of Bars	Bar Size	Spacing c/c (in.)	Case 1	Case 2	Case 3	Case 4	
3′-0	10½	10	3	#4	14	#5				16.8
3′-6	10½	10	4	#4	11½	#5				22.9
4′-0	10½	10	5	#4	10	#5				29.9
4′-6	10½	10	7	#4	7½	#5				37.8
5′-0	11	10	6	#5	10½	#6				46.6
5′-6	12	10	7	#5	9½	#6	#5			56.0
6′-0	13	10	6	#6	12½	#7	#6			66.2
6′-6	14	10	7	#6	11½	#8	#7	#5		77.1
7′-0	15	10	8	#6	11	#9	#7	#5		88.8
7′-6	15	12	7	#7	13½	#9	#7	#5		102.0
8′-0	16	12	8	#7	12½	#9	#7	#6	#5	115.2
8′-6	17	12	7	#8	15½	#10	#8	#6	#5	129.2
9′-0	18	14	7	#8	16½	#11	#9	#7	#6	143.8
9′-6	18	14	8	#8	15	#11	#9	#7	#6	160.2
10′-0	19	14	9	#8	14	#11	#9	#7	#6	176.3
10′-6	20	14	8	#9	16½	#11	#10	#8	#6	192.9
11′-0	20	16	9	#9	15½	#11	#10	#8	#6	211.7
11′-6	21	16	10	#9	14½	#11	#10	#8	#7	229.8
12′-0	22	16	10	#9	15	#11	#11	#9	#7	248.4

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

For explanation and limitations of these tables, see pages 366 to 368.

SOIL PRESSURE—3000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way			Max. Size of Dowels (see explanation on page 366)				Capacity (kips)
			Quant. of Bars	Bar Size	Spacing c/c (in.)	Case 1	Case 2	Case 3	Case 4	
3′-0	10½	10	4	#4	9½	#5				25.8
3′-6	10½	10	6	#4	7	#5				35.1
4′-0	10½	10	7	#4	6½	#5				45.9
4′-6	11	10	9	#4	5¾	#6				58.0
5′-0	13	10	7	#5	8½	#7	#6			70.9
5′-6	14	10	8	#5	8¼	#8	#6			85.4
6′-0	14	12	8	#6	9	#8	#6			101.7
6′-6	16	12	8	#6	10	#9	#7	#6	#5	118.3
7′-0	17	12	7	#7	12½	#10	#8	#6	#5	136.6
7′-6	17	14	8	#7	11½	#10	#8	#6	#5	156.8
8′-0	18	14	10	#7	9½	#11	#9	#7	#6	177.6
8′-6	19	16	8	#8	13¼	#11	#9	#7	#6	199.6
9′-0	20	16	7	#9	16½	#11	#10	#8	#6	222.7
9′-6	21	16	8	#9	15	#11	#10	#8	#7	247.1
10′-0	22	16	9	#9	13¾	#11	#11	#9	#7	272.5
10′-6	23	16	10	#9	13	#11	#11	#9	#8	299.1
11′-0	24	16	11	#9	12¼	#11	#11	#9	#8	326.7
11′-6	25	16	12	#9	11¾	#11	#11	#10	#8	355.5
12′-0	26	16	13	#9	11¼	#11	#11	#10	#9	385.2

SOIL PRESSURE—4000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way			Max. Size of Dowels (see explanation on page 366)				Capacity (kips)
			Quant. of Bars	Bar Size	Spacing c/c (in.)	Case 1	Case 2	Case 3	Case 4	
3′-0	10½	10	5	#4	7	#5				34.8
3′-6	10½	10	7	#4	5½	#5				47.4
4′-0	11	10	9	#4	5	#6				61.8
4′-6	13	10	10	#4	5	#7	#6			77.7
5′-0	14	10	8	#5	7¼	#8	#6	#5		95.6
5′-6	15	12	9	#5	7¼	#9	#7	#5		115.3
6′-0	16	12	8	#6	9	#9	#7	#6	#5	136.8
6′-6	17	14	9	#6	8¾	#10	#8	#6	#5	160.0
7′-0	18	14	11	#6	7½	#11	#9	#7	#6	185.0
7′-6	19	16	9	#7	10	#11	#9	#7	#6	211.7
8′-0	20	16	11	#7	8¾	#11	#10	#8	#6	240.0
8′-6	21	16	10	#8	10¼	#11	#10	#8	#7	270.1
9′-0	22	16	9	#9	12¼	#11	#11	#9	#7	301.7
9′-6	24	16	9	#9	13	#11	#11	#9	#8	333.9
10′-0	25	16	10	#9	12¼	#11	#11	#10	#8	368.8
10′-6	26	16	12	#9	10½	#11	#11	#10	#9	405.2
11′-0	27	16	13	#9	10¼	#11	#11	#11	#9	443.2
11′-6	28	16	14	#9	10	#11	#11	#11	#9	482.7
12′-0	29	18	15	#9	9½	#11	#11	#11	#10	523.9

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

For explanation and limitations of these tables, see pages 366 to 368.

SOIL PRESSURE—5000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way: Quant. of Bars	Mat Bars Each Way: Bar Size	Mat Bars Each Way: Spacing c/c (in.)	Max. Size of Dowels (see explanation on page 366): Case 1	Case 2	Case 3	Case 4	Capacity (kips)
3′-0	10½	10	6	#4	5½	#5				43.8
3′-6	10½	10	9	#4	4¼	#5				59.6
4′-0	12	10	10	#4	4½	#6	#5			77.6
4′-6	14	10	9	#5	5¾	#8	#6	#5		97.7
5′-0	15	12	9	#5	6½	#9	#7	#5		120.3
5′-6	15	14	11	#5	5¾	#9	#7	#5		145.6
6′-0	17	14	9	#6	8	#10	#8	#6	#5	172.4
6′-6	18	14	11	#6	7	#11	#9	#7	#6	201.7
7′-0	20	14	12	#6	6¾	#11	#10	#8	#6	232.7
7′-6	21	14	11	#7	8	#11	#10	#8	#7	266.5
8′-0	22	14	13	#7	7¼	#11	#11	#9	#7	302.4
8′-6	23	16	11	#8	9¼	#11	#11	#9	#8	340.5
9′-0	24	16	12	#8	9	#11	#11	#9	#8	380.7
9′-6	26	16	13	#8	8¾	#11	#11	#10	#9	421.9
10′-0	27	16	12	#9	10	#11	#11	#11	#9	466.3
10′-6	28	18	13	#9	9¾	#11	#11	#11	#9	512.7
11′-0	29	18	14	#9	9½	#11	#11	#11	#10	561.2
11′-6	31	18	12	#10	11¾	#11	#11	#11	#10	610.1
12′-0	31	20	13	#10	11¼	#11	#11	#11	#10	664.3

SOIL PRESSURE—6000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way: Quant. of Bars	Mat Bars Each Way: Bar Size	Mat Bars Each Way: Spacing c/c (in.)	Max. Size of Dowels (see explanation on page 366): Case 1	Case 2	Case 3	Case 4	Capacity (kips)
3′-0	10½	10	8	#4	4	#5				52.8
3′-6	12	10	9	#4	4¼	#6	#5	#5		71.7
4′-0	13	10	11	#4	4	#7	#6	#6		93.4
4′-6	14	12	10	#5	5	#8	#6	#6		117.9
5′-0	16	14	10	#5	5¾	#9	#7	#7	#5	145.0
5′-6	17	14	11	#5	5¾	#10	#8	#8	#5	175.1
6′-0	18	14	11	#6	6¼	#11	#9	#9	#6	207.9
6′-6	20	14	12	#6	6¼	#11	#10	#10	#6	242.9
7′-0	21	14	11	#7	7½	#11	#10	#10	#7	281.2
7′-6	22	14	12	#7	7½	#11	#11	#11	#7	322.0
8′-0	23	16	11	#8	8¾	#11	#11	#11	#8	365.6
8′-6	25	16	12	#8	8½	#11	#11	#11	#8	410.9
9′-0	26	16	11	#9	10	#11	#11	#11	#9	459.7
9′-6	27	18	12	#9	9½	#11	#11	#11	#9	511.1
10′-0	28	18	13	#9	9¼	#11	#11	#11	#9	565.0
10′-6	29	20	14	#9	9	#11	#11	#11	#10	621.6
11′-0	31	20	12	#10	11¼	#11	#11	#11	#10	679.2
11′-6	32	20	14	#10	10	#11	#11	#11	#11	740.6
12′-0	33	22	15	#10	9¾	#11	#11	#11	#11	804.7

SAFE CARRYING CAPACITY OF SQUARE INDIVIDUAL COLUMN FOOTINGS

For explanation and limitations of these tables, see pages 366 to 368.

SOIL PRESSURE—8000 psf

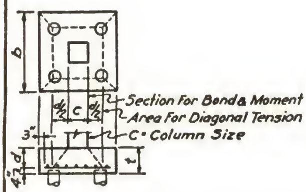
Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way: Quant. of Bars	Bar Size	Spacing c/c (in.)	Max. Size of Dowels (see explanation on page 366): Case 1	Case 2	Case 3	Case 4	Capacity (kips)
3′-0	11	10	10	#4	3¼	#6				70.8
3′-6	13	10	11	#4	3½	#7	#6			96.0
4′-0	14	12	12	#4	3¾	#8	#6	#5		125.2
4′-6	15	14	11	#5	4½	#9	#7	#5		158.2
5′-0	16	14	13	#5	4¼	#9	#8	#6	#5	195.0
5′-6	18	14	12	#6	5¼	#11	#9	#7	#6	235.2
6′-0	20	14	13	#6	5¼	#11	#10	#8	#6	279.0
6′-6	21	14	12	#7	6¼	#11	#10	#8	#7	326.9
7′-0	22	16	13	#7	6¼	#11	#11	#9	#7	378.5
7′-6	24	16	12	#8	7¼	#11	#11	#9	#8	433.1
8′-0	25	18	13	#8	7¼	#11	#11	#10	#8	492.0
8′-6	27	18	14	#8	7¼	#11	#11	#11	#9	553.6
9′-0	28	20	13	#9	8¼	#11	#11	#11	#9	619.6
9′-6	29	20	14	#9	8	#11	#11	#11	#10	689.3
10′-0	31	20	15	#9	8	#11	#11	#11	#10	761.3
10′-6	32	22	13	#10	9¾	#11	#11	#11	#11	837.9
11′-0	34	22	14	#10	9½	#11	#11	#11	#11	916.6
11′-6	35	24	15	#10	9¼	#11	#11	#11	#11	1000.2
12′-0	35	26	17	#10	8½	#11	#11	#11	#11	1089.1

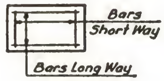
SOIL PRESSURE—10,000 psf

Size B	Thickness t (in.)	Min. Col. Size (in.)	Mat Bars Each Way: Quant. of Bars	Bar Size	Spacing c/c (in.)	Max. Size of Dowels (see explanation on page 366): Case 1	Case 2	Case 3	Case 4	Capacity (kips)
3′-0	12	10	9	#5	3½	#6	#5			88.6
3′-6	13	12	10	#5	3¾	#7	#6			120.5
4′-0	14	14	12	#5	3½	#8	#6	#5		157.2
4′-6	16	14	13	#5	3¾	#9	#7	#6	#5	198.4
5′-0	17	14	15	#5	3¾	#10	#8	#6	#5	244.7
5′-6	19	14	14	#6	4½	#11	#9	#7	#6	295.3
6′-0	20	16	15	#6	4½	#11	#10	#8	#6	351.0
6′-6	22	16	14	#7	5¼	#11	#11	#9	#7	410.9
7′-0	24	16	15	#7	5¼	#11	#11	#9	#8	475.3
7′-6	25	18	16	#7	5½	#11	#11	#10	#8	544.9
8′-0	26	20	14	#9	6¾	#11	#11	#10	#9	619.2
8′-6	28	20	14	#9	7¼	#11	#11	#11	#9	697.2
9′-0	29	22	15	#9	7	#11	#11	#11	#10	780.7
9′-6	31	22	16	#9	7	#11	#11	#11	#10	867.6
10′-0	32	24	17	#9	7	#11	#11	#11	#11	960.0
10′-6	34	24	15	#10	8¼	#11	#11	#11	#11	1055.6
11′-0	35	26	16	#10	8¼	#11	#11	#11	#11	1157.1
11′-6	36	28	18	#10	7½	#11	#11	#11	#11	1263.0
12′-0	38	28	19	#10	7½	#11	#11	#11	#11	1371.6

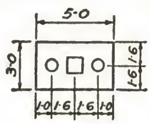
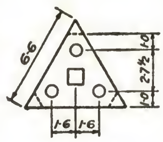
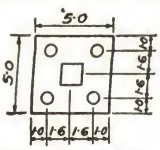
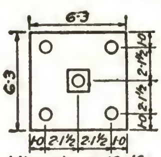
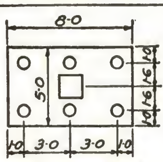
CONCRETE PILE FOOTINGS

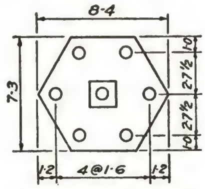
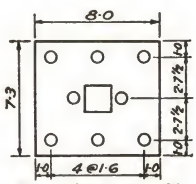
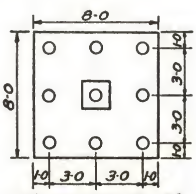
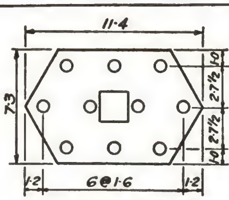
30-Ton Piles 3′-0″ c. to c.*





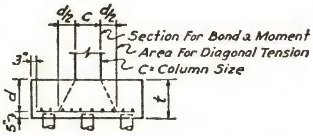
f_s = 20,000 psi n = 10
f_c = 3000 psi v = 75 psi
f'_c = 1350 psi u = 240 psi

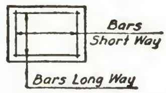
No. Piles	PLAN	Column Load (kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
2	 Min. column 12x12	115	18/22	4-#5	8-#5
			22/26	4-#5	7-#5
			24/28	3-#5	6-#5
3	 Min. column 12x12	174	18/22	3 ways ea. of: 10-#5	
			21/25	8-#5	
			24/28	7-#5	
4	 Min. column 16x16	231	17/21	17-#5	17-#5
			20/24	15-#5	15-#5
			24/28	12-#5	12-#5
5	 Min. column 16x16	286	23/27	17-#5	17-#5
			26/30	15-#5	15-#5
			30/34	13-#5	13-#5
6	 Min. column 18x18	343	26/30	17-#5	16-#6
			30/34	15-#5	14-#6
			34/38	13-#5	13-#6

No. Piles	PLAN	Column Load (kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
7	 Min. column 20x20	396	28/32	12-#6	13-#6
			32/36	11-#6	11-#6
			34/38	10-#6	10-#6
8	 Min. column 20 x 20	452	30/34	17-#6	16-#6
			34/38	15-#6	14-#6
			36/40	14-#6	13-#6
9	 Min. column 22 x 22	506	34/38	17-#6	17-#6
			38/42	16-#6	16-#6
			42/46	14-#6	14-#6
10	 Min. column 24x24	562	37/41	18-#5	15-#7
			40/44	16-#5	14-#7
			44/48	15-#5	13-#7

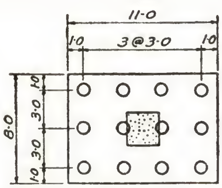
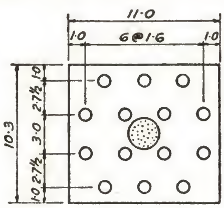
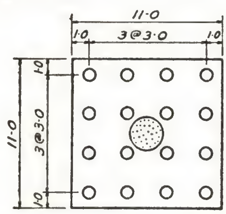
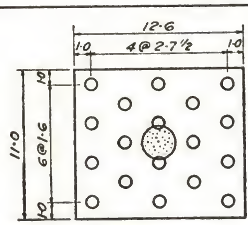
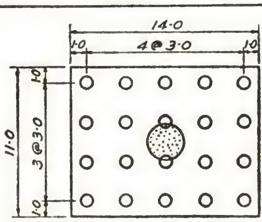
* Piles are here assumed as carrying 30 tons each, with no reduction for the effect of neighboring piles in the cluster.

CONCRETE PILE FOOTINGS
30-Ton Piles 3′-0″ c. to c.*

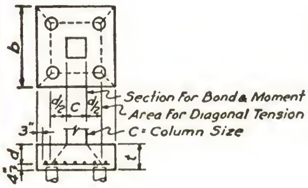




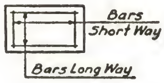
f_s = 20,000 psi n = 10
f'_c = 3,000 psi v = 75 psi
f_c = 1,350 psi u = 240 psi

No. Piles	PLAN		Column Load (Kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
12		Min. Column 27 x 27	676	35/40	10-#8	19-#7
			668	42/47	8-#8	16-#7
			662	48/53	10-#7	14-#7
14		Min. Column 24″ ϕ	781	37/42	20-#7	20-#7
			772	43/48	17-#7	18-#7
			765	48/53	10-#9	16-#7
16		Min. Column 26″ ϕ	890	41/46	18-#8	18-#8
			883	46/51	21-#7	21-#7
			874	52/57	11-#9	11-#9
18		Min. Column 26″ ϕ	992	46/51	16-#8	22-#8
			982	52/57	14-#8	15-#9
			972	58/63	17-#7	17-#8
20		Min. Column 28″ ϕ	1100	47/52	19-#8	27-#8
			1086	54/59	17-#8	23-#8
			1075	60/65	15-#8	17-#9

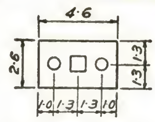
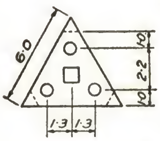
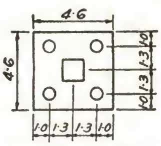
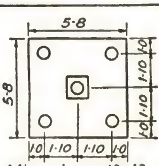
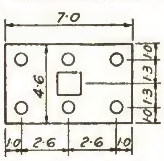
* Piles are here assumed as carrying 30 tons each, with no reduction for the effect of neighboring piles in the cluster.

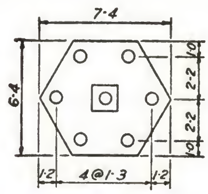
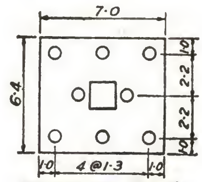
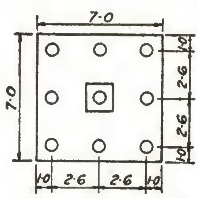
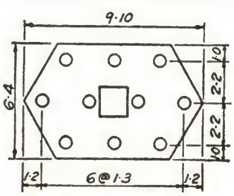


WOOD PILE FOOTINGS
15-Ton Piles 2′-6″ c. to c.*



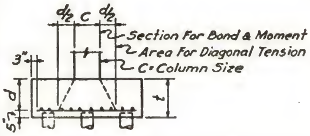
f_s = 20,000 psi n = 10
f'_c = 3000 psi v = 75 psi
f_c = 1350 psi u = 240 psi

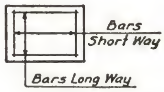
No. Piles	PLAN	Column Load (kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
2	 Min. column 12x12	58	9/13	4-#5	8-#5
			12/16	3-#5	6-#5
			15/19	3-#5	5-#5
3	 Min. column 12x12	86	12/16	3 ways ea. of: 7-#5	
			15/19	6-#5	
			18/22	5-#5	
4	 Min. column 12x12	115	13/17	12-#5	12-#5
			15/19	10-#5	10-#5
			18/22	8-#5	8-#5
5	 Min. column 13x13	141	15/19	12-#5	12-#5
			18/22	10-#5	10-#5
			20/24	9-#5	9-#5
6	 Min. column 14x14	171	18/22	12-#5	10-#6
			20/24	11-#5	9-#6
			23/27	10-#5	8-#6

No. Piles	PLAN	Column Load (kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
7	 Min. column 15 x 15	199	19/23	11-#5	12-#5
			21/25	10-#5	11-#5
			24/28	9-#5	9-#5
8	 Min. column 16 x 16	226	18/22	12-#6	12-#6
			21/25	10-#6	10-#6
			24/28	9-#6	9-#6
9	 Min. column 17 x 17	253	21/25	14-#6	14-#6
			23/27	13-#6	13-#6
			26/30	10-#6	10-#6
10	 Min. column 18 x 18	281	23/27	14-#5	12-#7
			26/30	12-#5	11-#7
			30/34	11-#5	9-#7

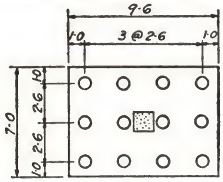
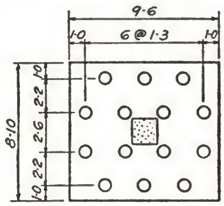
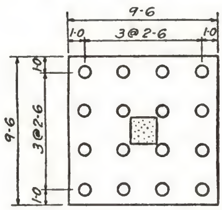
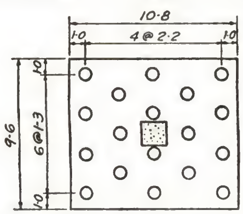
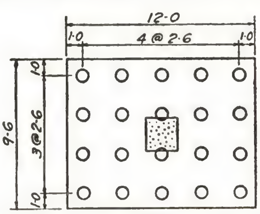
* Piles are here assumed as carrying 15 tons each, with no reduction for the effect of neighboring piles in the cluster.

WOOD PILE FOOTINGS
15-Ton Piles 2′-6″ c. to c.*





f_s = 20,000 psi n = 10
f'_c = 3,000 psi v = 75 psi
f_c = 1,350 psi u = 240 psi

No. Piles	PLAN		Column Load (Kips)	d and t (in.)	Reinforcement Short way	Reinforcement Long way
12		Min. Column 19 x 19	332	25/30	11-#6	12-#7
				28/33	10-#6	11-#7
				32/37	12-#5	13-#6
14		Min. Column 20 x 20	385	25/30	13-#7	12-#7
				28/33	11-#7	11-#7
				32/37	12-#6	13-#6
16		Min. Column 22 x 22	440	27/32	14-#7	14-#7
				30/35	13-#7	13-#7
				34/39	11-#7	11-#7
18		Min. Column 22 x 22	488	31/36	12-#7	17-#7
				36/41	11-#7	11-#8
				40/45	10-#7	13-#7
20		Min. Column 24 x 24	541	32/37	14-#7	13-#9
				36/41	17-#6	14-#8
				40/45	15-#6	17-#7

* Piles are here assumed as carrying 15 tons each, with no reduction for the effect of neighboring piles in the cluster.

ALLOWABLE CONCENTRIC LOADS ON STEEL PIPE COLUMNS

STANDARD

Unbraced Length (ft)	Nominal Diameter—Weight Per Foot											
	12		10			8		6	5	4	3½	3
	49.56	43.77	40.48	34.24	31.20	28.55	24.70	18.97	14.62	10.79	9.11	7.58
6	246	217	200	169	154	140	121	92	70	50	42	33
8	244	216	199	168	153	138	120	90	68	47	38	30
10	243	214	196	166	151	136	118	86	64	44	35	26
12	240	212	194	164	149	133	115	82	61	40	30	21
14	237	210	190	161	147	129	112	79	56	34	25	18
16	234	207	187	158	144	125	109	74	51	30	22	16
18	231	204	182	154	141	121	105	69	45	26	19	13
20	227	200	178	151	137	115	100	63	41	23	17	
22	222	196	172	146	133	109	95	56	37	21	15	

EXTRA STRONG

Unbraced Length (ft)	Nominal Diameter—Weight Per Foot							
	12	10	8	6	5	4	3½	3
	65.42	54.74	43.39	28.57	20.78	14.98	12.51	10.25
6	325	271	213	139	99	70	58	45
8	323	268	210	135	96	65	53	40
10	320	265	206	131	91	60	47	35
12	317	261	201	125	85	54	40	28
14	313	257	196	119	79	47	34	24
16	309	252	189	112	71	40	30	21
18	304	246	182	103	63	36	26	18
20	299	239	173	94	56	32	23	
22	293	232	164	84	51	28		
24	286	224	155	77	46	25		

DOUBLE EXTRA STRONG

Unbraced Length (ft)	Nominal Diameter—Weight Per Foot					
	8	6	5	4	3½	3
	72.42	53.16	38.55	27.54	22.85	18.58
6	355	257	183	130	103	80
8	350	249	176	118	93	70
10	343	240	165	108	82	59
12	334	228	154	94	68	48
14	324	213	140	79	58	40
16	312	200	125	70	50	34
18	299	182	109	61	43	
20	284	163	98	54	38	
22	269	147	88	47		
24	250	135	80			
26	230	124	72			

For concrete-filled pipe columns, see page 378.

Loads below heavy line are for secondary members with L/r ratios between 120 and 200. Properties of steel from which pipe is made are assumed to be those of ASTM A7. If pipe is made of other steel, safe loads should be suitably modified.

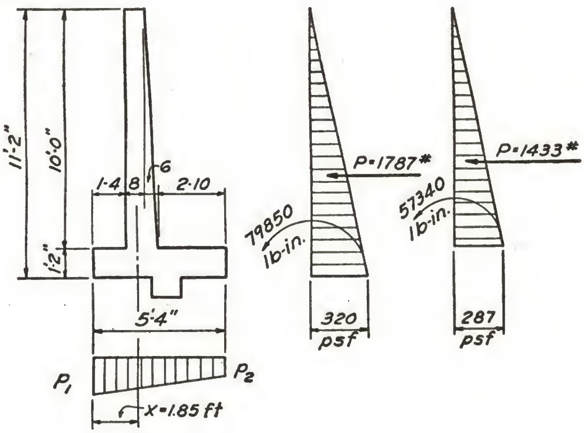
CANTILEVERED RETAINING WALLS

The walls tabulated here are designed to have a factor of safety against overturning varying from a little less than 2 to a little over 2½, with a toe pressure not exceeding 4000 psf (except in the case of walls over 15 ft high with sloping backfill when the earth pressure runs up to 5000 psf). The passive resistance of any earth at the toe of the wall has been neglected. Generally, and especially in moist clay soils, a lug or key projecting down below the main footing level is desirable to assist in preventing sliding.

While these tables cover quite a wide range, it may be necessary to compute values beyond the scope of the book, or someone may want to see how the tabulated values were worked out, so the following example is included:— *

Example—For the table on page 384, prepare the design of a cantilevered retaining wall 10 feet high from top of footing to top of wall, the surface of the earth fill being level at the top of the wall with no surcharge.

The outlines of the concrete, batter, and the location of the stem on the footing are matters of experience and of trial, quite a few proportions having been investigated before arriving at the values given. However, the checking of the tabulated values is fairly simple, taking concrete at 150 pcf and backfill at 100 pcf, with $\phi = 33°40'$, stresses being as tabulated at the head of the table.



Stability—

Resisting Moment—

		W	× x	= M
Footing:—	1.17×5.33 @ 150 =	935 lb	2.67 ft =	2,497 lb-ft
Stem:—	0.67×10.00 @ 150 =	1005 lb	1.67 ft =	1,678 lb-ft
	$0.50 \times \frac{10.00}{2}$ @ 150 =	375 lb	2.17 ft =	814 lb-ft
Earth:—	2.83×10.00 @ 100 =	2830 lb	3.92 ft =	11,094 lb-ft
	$0.50 \times \frac{10.00}{2}$ @ 100 =	250 lb	2.33 ft =	582 lb-ft
		5395 lb		16,665 lb-ft

Overturning Moment (from the table on page 394):—

$0.5734\, wh^3 = 0.5734 \times 100 \times 11.17^3 = 79{,}850$ lb-in. $= -\ 6{,}654$ lb-ft

5395)10,011

Resultant base pressure acts at $x = 1.85$ ft from toe.

* For a more extended treatment of retaining wall design, see Sutherland and Reese "Reinforced Concrete Design," John Wiley & Sons, Inc., 1943.

CANTILEVERED RETAINING WALLS

Factor against Overturning— $\dfrac{16{,}665}{6{,}654} = 2.50$ (as in table, page 384)

Pressure on Soil—Eccentricity of resultant pressure on base =

$\dfrac{5.33}{2} - 1.85 = 0.82 \text{ ft} < \dfrac{5.33}{6}$. So the force lies within the middle third of the base.

$$p_1 = \frac{P}{A} \pm \frac{Mc}{I} = \frac{P}{bh} + \frac{6Pe}{bh^2} = \frac{5395}{5.33} + \frac{6 \times 5395 \times 0.82}{1 \times (5.33)^2}$$

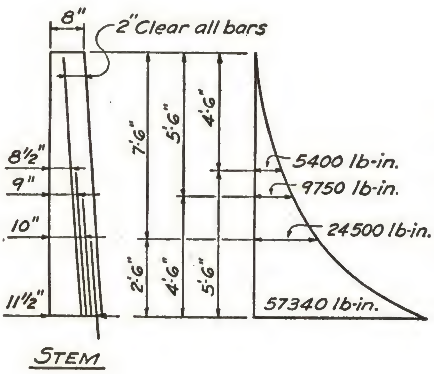
$$\left.\begin{aligned} &= 1012 + 932 = 1944 \text{ psf} \\ p_2 &= 1012 - 932 = 80 \text{ psf} \end{aligned}\right\} \text{In table, p. 385}$$

Stem

Shear—From the table on page 394, the horizontal thrust for $h = 10$ is 1433 lb.

$$v = \frac{V}{bjd} = \frac{1433}{12 \times \frac{7}{8} \times 11\frac{1}{2}} = 12 \text{ psi} < 90 \text{ psi}$$

Moment—The bending moment increases rapidly (as the cube of the height) from top to bottom of stem and the effective depth of the reinforcing steel also increases somewhat. In the following figure, the required amount of reinforcement is computed at several levels and a curve of required A_s is drawn. Economy results by



$$A_s = \frac{5400}{17500 \times 8\frac{1}{2}} = 0.037 \qquad \#3 @ 24'' = 0.055$$

$$A_s = \frac{9750}{17500 \times 9} = 0.062 \qquad \#3 @ 16'' = 0.083$$

$$A_s = \frac{24500}{17500 \times 10} = 0.140 \qquad \#3 @ 8'' = 0.165$$

$$A_s = \frac{57340}{17500 \times 11\frac{1}{2}} = 0.285 \qquad \#4 @ 8'' = 0.300$$

selecting dowels from the footing into the stem that are of proper size, spacing and length to take care of the peak of the curve for required A_s, while, at the top of these dowels, less reinforcement is necessary and only a portion of this need extend to the top of the wall. The amount of steel is computed in the above figure. The spacings of bars in the wall and footing are kept uniform to produce a simple pattern for the erectors.

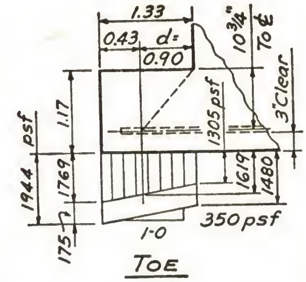
Bond—On dowels:—$u = \dfrac{vs}{\Sigma_0} = \dfrac{12 \times 8}{1.571} = 61 \text{ psi} < 300 \text{ psi}$

Bond on vertical bars at various levels should be similarly investigated.

CANTILEVERED RETAINING WALLS

Toe

Loads—Concrete:—$1.17 \times 150 = 175$ psf
At toe:—$1944 - 175 = 1769$ psf
Under face of stem:—$1480 - 175 = 1305$ psf



Shear—ACI 1205a permits computing shear on a plane at distance, d, from the face of the wall, where

$$V = 0.43\frac{1769 + 1619}{2} = 728 \text{ lb}$$

$$v = \frac{V}{bjd} = \frac{728}{12 \times \frac{7}{8} \times 10\frac{3}{4}} = 7 \text{ psi} < 90 \text{ psi}$$

More conservatively, shear can be computed on the face of the wall

$$V = 1.38\frac{1769 + 1305}{2} = 2044 \text{ lb}$$

$$v = \frac{V}{bjd} = \frac{2044}{12 \times \frac{7}{8} \times 10\frac{3}{4}} = 18 \text{ psi} < 90 \text{ psi}$$

For this wall, either value is well on the conservative side.

Moment about Face of Stem—$M = 1305 \times 1.33 \times 0.67 = 1163$ lb-ft

$$464 \times \frac{1.33}{2} \times \frac{2 \times 1.33}{3} = +\underline{276} \text{ lb-ft}$$

$$1439 \text{ lb-ft} = 17{,}300 \text{ lb-in.}$$

$$A_s = \frac{M}{f_s jd} = \frac{17{,}300}{20{,}000 \times \frac{7}{8} \times 10\frac{3}{4}} = 0.092 \text{ sq in.}$$

#4 @ 24 = 0.10 sq in.

Bond—Bond is to be computed at the face of the wall, ACI 1205-d

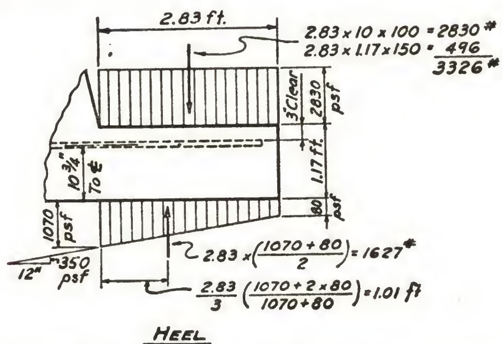
$$u = \frac{vs}{\Sigma_o} = \frac{18 \times 24}{1.571} = 275 \text{ psi} < 300 \text{ psi}$$

Heel

Shear—While ACI 1205a would seem to permit computing shear on a plane at distance, d, from the back of the wall, it is more conservative and probably desirable to compute on a plane at the back of the wall

$$V = 3326 - 1627 = 1699 \text{ lb}$$

$$v = \frac{V}{bjd} = \frac{1699}{12 \times \frac{7}{8} \times 10\frac{3}{4}} = 15.1 \text{ psi} < 90 \text{ psi}$$



$$M \text{ about back of stem} = 3326 \times \frac{2.83}{2} = \quad 4700 \text{ lb-ft}$$

$$1627 \times 1.01 = -\underline{1643} \text{ lb-ft}$$

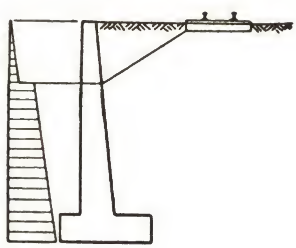
$$3057 \text{ lb-ft} = 36{,}700 \text{ lb-in.}$$

$$A_s = \frac{M}{f_s jd} = \frac{36{,}700}{20{,}000 \times 7/8 \times 10\frac{3}{4}} = 0.195 \text{ sq in.}$$

#5 @ 16 = 0.23 sq in. (from the table on page 384).

$$Bond\text{—}u = \frac{vs}{\Sigma_o} = \frac{15.1 \times 16}{1.963} = 123 \text{ psi} < 300 \text{ psi}$$

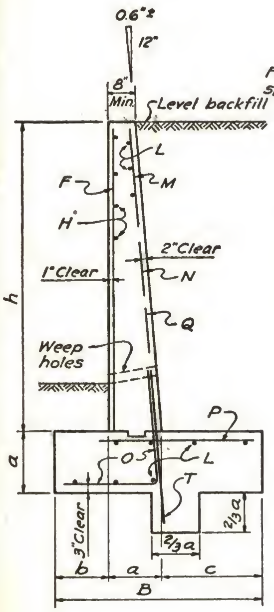
The designs in the other tables follow the same procedure. It is unnecessary to work out complete examples. A few observations will be helpful. For a sloping backfill, the resultant pressure is parallel to the slope; its vertical component was included in computing overturning and resisting moments. In the case of the highway and railroad surcharges, it was assumed that the increased intensity of thrust was effective the full height of the wall. If the line of travel is established somewhat back from the stem, it is possible to draw a sloping pressure line down from the nearest edge of the load to the back of the wall and omit the increased intensity above the intersection. (See the figure below.)

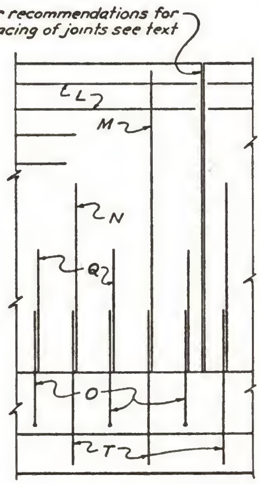


The case of an adjoining building exercising a thrust directly against the back of the wall is one requiring special study. Roughly, such a wall would work out about as much heavier than the case of a railroad surcharge as that case is heavier than a wall with an ordinary horizontal backfill.

In all of these tables, the top of the wall is arbitrarily taken as 8 in. thick. This is about the minimum through which proper concrete can be cast with all of the reinforcement in place. For the higher walls, some authorities recommend at least a 12 in. thickness. The user may increase the top thickness without changing the bottom thickness or reinforcing steel, if he cares to do so, the increased weight adding slightly to the resistance to overturning.

CANTILEVERED RETAINING WALLS—BACKFILL LEVEL—NO SURCHARGE





Vertical steel in back of wall may be:—

$\left\{\begin{array}{l} O + T \text{ only} \\ O \text{ only} \\ M + N \text{ alt.} \end{array}\right. \quad \left.\begin{array}{l} M + Q + N + Q \\ \text{(as shown)} \end{array}\right\}$

Comparison of bar spacings in table will indicate combination, spacings being accumulative from a selected M or O bar.

Wt. of earth = 100 pcf
∠ of Internal Friction = ϕ = 33° 40′
f_s = 20,000 psi
f'_c = 3,000 psi
f_c = 1,350 psi
v = 90 psi
u = 300 psi

O & *T* bars alternate, or occur O + *T* + *T*, except in walls without key, where only O bars are required.

REINFORCEMENT

Height of Wall = h (ft)	M			N			Q			P			O		
	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)
3	—	—	—	—	—	—	—	—	—	#3	1′-3	18	#3	4′-3	18
4	—	—	—	—	—	—	—	—	—	#3	1′-3	18	#3	5′-5	18
5	—	—	—	—	—	—	—	—	—	#3	1′-9	18	#3	6′-5	18
6	—	—	—	—	—	—	—	—	—	#3	2′-3	16	#3	7′-10	16
7	—	—	—	—	—	—	—	—	—	#3	2′-6	10	#3	9′-2	10
8	#3	7′-10	22½	#3	3′-6	7½, 15	—	—	—	#4	2′-9	15	#3	5′-1	22½
9	#3	8′-10	16½	#3	4′-6	5½, 11	—	—	—	#4	3′-3	11	#3	3′-10	16½
10	#3	9′-10	16	#3	5′-6	16	—	—	—	#5	3′-6	16	#4	5′-6	24
√ 11	#3	10′-10	24	#3	6′-0	24	#3	5′-0	12	#4	4′-0	12	#4	5′-8	18
12	#3	11′-10	20	#3	7′-0	20	#3	5′-0	10	#5	4′-3	10	#4	6′-6	10
13	#4	12′-10	26	#4	7′-0	26	#4	5′-0	13	#4	4′-6	6½	#5	6′-2	19½
14	#4	13′-10	20	#4	7′-6	20	#4	5′-0	10	#4	4′-9	5	#5	6′-3	15
15	#5	14′-10	26	#5	8′-0	26	#5	6′-0	13	#5	5′-3	6½	#6	7′-3	19½
16	#5	15′-10	22	#5	8′-0	22	#5	6′-0	11	#5	5′-9	5½	#6	6′-10	16½
17	#6	16′-10	26	#6	8′-6	26	#6	6′-6	13	#6	6′-3	6½	#7	7′-6	13
18	#6	17′-10	24	#6	9′-6	24	#6	7′-0	12	#6	6′-6	6	#7	7′-9	12
19	#7	18′-10	26	#7	10′-0	26	#7	6′-6	13	#7	7′-3	6½	#8	8′-4	19½
20	#7	19′-10	24	#7	10′-6	24	#7	7′-0	12	#6	7′-3	6	#8	8′-6	24

CANTILEVERED RETAINING WALLS—BACKFILL LEVEL—NO SURCHARGE

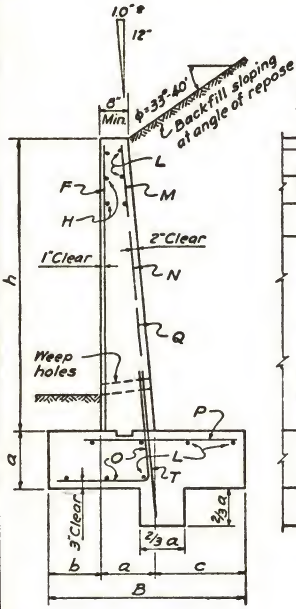
CONCRETE OUTLINES

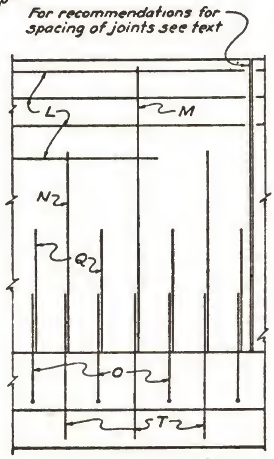
Height of Wall = h (ft)	B (ft)	a (ft)	b (ft)	c (ft)	Base Pressure at Toe (psf)	Base Pressure at Heel (psf)	Resisting Moment (lb-ft)	Overturning Moment (lb-ft)	$\frac{M_R}{M_O}$	Concrete (cu ft per lin. ft of wall)
3	1′-9	0′-10½	0′-4	0′-6½	885	0	726	279	2.60	4.2
4	2′-2	0′-11	0′-5	0′-10	1,136	0	1,359	570	2.38	5.5
5	2′-8	0′-11	0′-5	1′-4	1,361	0	2,390	991	2.41	6.7
6	3′-3	1′-0	0′-8	1′-7	1,415	0	4,100	1,638	2.50	8.7
7	3′-10	1′-0	1′-0	1′-10	1,398	89	6,300	2,448	2.58	10.0
8	4′-3	1′-1	1′-1	2′-1	1,643	38	8,760	3,582	2.44	11.9
9	4′-9	1′-1	1′-1	2′-7	1,851	60	12,310	4,892	2.51	12.5
10	5′-4	1′-2	1′-4	2′-10	1,944	80	16,665	6,675	2.50	16.5
11	5′-10	1′-2	1′-6	3′-2	2,051	108	21,400	8,600	2.49	17.6
12	6′-6	1′-3	1′-8	3′-7	2,129	207	28,900	10,980	2.63	20.4
13	7′-0	1′-4	1′-8	4′-0	2,403	161	36,300	14,101	2.58	23.0
14	7′-8	1′-4	2′-1	4′-3	2,332	290	45,700	17,256	2.65	25.0
15	8′-1	1′-5	2′-1	4′-7	2,608	239	54,700	21,158	2.59	28.0
16	8′-6	1′-5	2′-2	4′-11	2,799	211	64,000	25,253	2.53	29.7
17	9′-0	1′-6	2′-3	5′-3	3,008	197	76,300	30,325	2.52	32.9
18	9′-6	1′-7	2′-4	5′-7	3,207	197	90,000	35,898	2.50	36.5
19	10′-2	1′-7	2′-6	6′-1	3,264	286	107,700	41,654	2.58	38.5
20	10′-5	1′-8	2′-6	6′-3	3,119	646	119,300	48,650	2.46	42.0

REINFORCEMENT

O a b		T			F			L			H			Weight of Bars (lb per lin. ft of wall)
a (ft)	b (ft)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Quant. of Bars	Bar Size	Spcg (in.)	Quant. of Bars	Bar Size	Spcg (in.)	
0′-10	3′-5	—	—	—	#3	2′-10	18	7	#3	12	2	#3	18	5.22
0′-11	4′-6	—	—	—	#3	3′-10	18	8	#3	12	3	#3	18	6.39
0′-11	5′-6	—	—	—	#3	4′-10	18	9	#3	12	3	#3	18	7.40
1′-3	6′-7	—	—	—	#3	5′-10	18	11	#3	12	4	#3	18	10.21
1′-7	7′-7	—	—	—	#3	6′-10	12	12	#3	12	7	#3	12	16.18
1′-9	3′-4	#3	2′-10	7½, 15	#3	7′-10	12	14	#3	12	8	#3	12	18.26
1′-9	2′-1	#3	2′-10	5½, 11	#3	8′-10	12	15	#3	12	9	#3	12	21.07
2′-1	3′-5	#4	3′-5	8, 16	#4	9′-10	12	17	#4	12	10	#4	12	35.8
2′-3	3′-5	#4	4′-3	6, 12	#4	10′-10	12	19	#4	12	11	#4	12	43.2
2′-6	4′-0	#4	4′-10	10	#4	11′-10	12	20	#4	12	12	#4	12	50.2
2′-7	3′-7	#5	4′-6	6½, 13	#4	12′-10	12	21	#4	12	13	#4	12	55.8
3′-0	3′-3	#5	4′-1	5, 10	#4	13′-10	12	23	#4	12	14	#4	12	66.2
3′-1	4′-2	#6	5′-2	6½, 13	#4	14′-10	12	25	#4	12	15	#4	12	79.7
3′-2	3′-8	#6	4′-8	5½, 11	#4	15′-10	12	27	#4	12	16	#4	12	90.5
3′-4	4′-2	#7	5′-2	13	#4	16′-10	12	28	#4	12	17	#4	12	108.9
3′-6	4′-3	#7	5′-4	12	#4	17′-10	12	29	#4	12	18	#4	12	120.7
3′-8	4′-8	#8	5′-9	6½, 13	#4	18′-10	12	30	#4	12	19	#4	12	144.9
3′-9	4′-9	#8	5′-11	6, 12, 18	#4	19′-10	12	31	#4	12	20	#4	12	157.3

CANTILEVERED RETAINING WALLS—SURFACE OF EARTH SLOPING ($\phi = 33°40'$)





Vertical steel in back of wall may be:

O + T only	M + N + 1Q
O only	M + Q + N + Q
M + N alt.	(as shown)

Comparison of bar spacings in table will indicate combination, spacings being accumulative from a selected M or O bar.

Wt of earth = 100 pcf
L of Internal Friction = ϕ = 33° 40′
f_s = 20,000 psi
f'_c = 3,000 psi
f_c = 1,350 psi
v = 90 psi
u = 300 psi

O & T bars alternate, or occur O + T + T except in walls without key, where only O bars are required.

REINFORCEMENT

Height of Wall = h (ft)	M			N			Q			P			O		
	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)
3	—	—	—	—	—	—	—	—	—	#3	1′-6	18	#3	4′-7	18
4	—	—	—	—	—	—	—	—	—	#3	2′-0	18	#3	5′-11	18
5	—	—	—	—	—	—	—	—	—	#3	2′-3	13	#3	7′-4	13
6	—	—	—	—	—	—	—	—	—	#3	2′-9	16	#3	8′-9	16
7	—	—	—	—	—	—	—	—	—	#3	3′-0	11	#3	10′-2	11
8	#4	7′-10	14	—	—	—	—	—	—	#4	3′-6	14	#4	6′-3	14
9	#4	8′-10	11	—	—	—	—	—	—	#5	4′-0	11	#4	6′-10	11
10	#4	9′-10	13	#4	4′-6	13	—	—	—	#5	4′-9	13	#5	6′-8	13
11	#5	10′-10	14	#5	5′-0	14	—	—	—	#4	4′-9	7	#6	7′-5	14
12	#6	11′-10	16	#6	5′-0	16	—	—	—	#6	5′-3	16	#7	7′-11	16
13	#5	12′-10	17	#5	5′-6	17	#5	4′-0	17	#5	5′-6	8½	#8	8′-10	17
14	#6	13′-10	15	#6	6′-0	15	#5	4′-0	15	#7	6′-6	15	#8	9′-3	15
15	#6	14′-10	18	#6	7′-6	18	#6	4′-6	18	#7	6′-9	12	#8	9′-8	12
16	#7	15′-10	19½	#7	7′-6	19½	#7	4′-6	19½	#8	7′-3	13	#9	10′-6	13
17	#7	16′-10	18	#7	9′-0	18	#7	5′-6	18	#8	7′-9	12	#9	11′-3	12
18	#7	17′-10	19½	#7	9′-0	19½	#8	5′-9	19½	#9	8′-3	13	#10	11′-9	13
19	#8	18′-10	16½	#8	9′-0	16½	#8	6′-0	16½	#9	8′-9	11	#10	12′-3	11
20	#9	19′-10	20	#9	10′-0	20	#9	6′-0	10	#9	9′-3	10	#10	12′-9	10

CANTILEVERED RETAINING WALLS—SURFACE OF EARTH SLOPING ($\phi = 33°40'$)

CONCRETE OUTLINES

Height of Wall = h (ft)	B (ft)	a (ft)	b (ft)	c (ft)	Base Pressure		Resisting Moment (lb-ft)	Overturning Moment (lb-ft)	$\frac{M_R}{M_O}$	Concrete (cf per lf of wall)
					At Toe (psf)	At Heel (psf)				
3	2′-6	0′-11	0′-7	1′-0	1,072	25	2,397	1,130	2.12	5.05
4	3′-2	1′-0	0′-9	1′-5	1,421	49	5,072	2,527	2.01	6.93
5	3′-10	1′-1	1′-0	1′-9	1,699	62	9,092	4,660	1.95	9.04
6	4′-6	1′-2	1′-3	2′-1	1,990	62	14,900	7,750	1.92	11.39
7	5′-3	1′-3	1′-6	2′-6	2,238	105	23,400	12,100	1.93	13.97
8	5′-11	1′-4	1′-9	2′-10	2,512	118	33,600	17,600	1.91	16.7
9	6′-8	1′-5	2′-0	3′-3	2,753	176	47,900	25,000	1.92	19.8
10	7′-5	1′-6	2′-3	3′-8	3,018	210	65,800	34,300	1.92	23.0
11	8′-1	1′-7	2′-6	4′-0	3,298	210	85,300	44,900	1.90	26.3
12	8′-10	1′-8	2′-9	4′-5	3,536	266	111,000	57,900	1.91	30.0
13	9′-6	1′-9	3′-0	4′-9	3,805	286	139,000	72,800	1.90	33.7
14	10′-3	1′-10	3′-3	5′-2	4,058	329	173,000	90,900	1.91	37.8
15	11′-0	1′-11	3′-6	5′-7	4,310	375	214,000	112,000	1.91	42.1
16	11′-10	2′-0	3′-10	6′-0	4,477	470	262,000	135,000	1.94	46.7
17	12′-7	2′-1	4′-1	6′-5	4,719	524	314,000	162,000	2.04	51.5
18	13′-4	2′-2	4′-4	6′-10	4,959	582	373,000	193,000	1.94	56.6
19	14′-2	2′-3	4′-8	7′-3	5,104	696	443,000	226,000	1.96	61.9
20	15′-0	2′-4	5′-0	7′-8	5,300	774	521,000	264,000	1.97	67.4

REINFORCEMENT

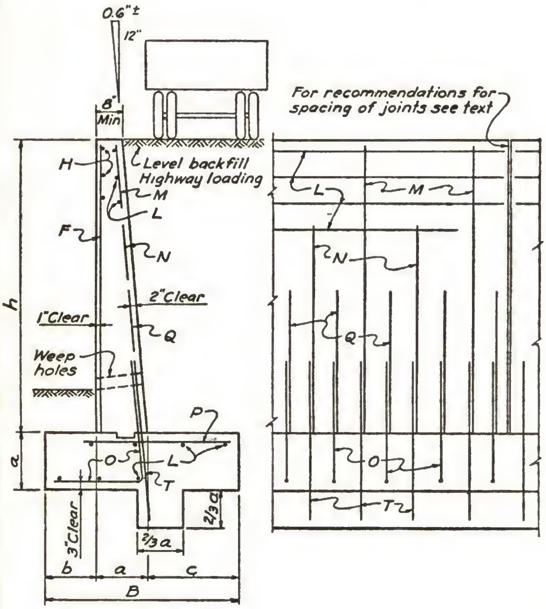
O a b		T			F			L			H			Weight of Bars (lb per lin. ft of wall)
a (ft)	b (ft)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Quant. of Bars	Bar Size	Spcg (in.)	Quant. of Bars	Bar Size	Spcg (in.)	
1′-1	3′-6	—	—	—	#3	2′-10	18	7	#3	12	2	#3	18	5.37
1′-4	4′-7	—	—	—	#3	3′-10	18	8	#3	12	3	#3	18	6.70
1′-8	5′-8	—	—	—	#3	4′-10	18	9	#3	12	3	#3	18	8.66
2′-0	6′-9	#3	4′-6	16	#3	5′-10	18	11	#3	12	4	#3	18	11.12
2′-4	7′-10	#3	4′-6	11	#3	6′-10	18	12	#3	12	7	#3	12	16.08
2′-8	3′-7	#4	4′-6	14	#3	7′-10	12	14	#3	12	8	#3	12	23.85
3′-0	3′-10	#4	4′-6	11	#3	8′-10	12	15	#3	12	9	#3	12	31.54
3′-4	3′-4	#5	4′-6	13	#4	9′-10	12	17	#4	12	10	#4	12	48.86
3′-8	3′-9	#6	5′-0	14	#4	10′-10	12	20	#4	12	11	#4	12	63.4
4′-0	3′-11	#7	5′-0	16	#4	11′-10	12	21	#4	12	12	#4	12	74.6
4′-4	4′-6	#8	5′-8	17	#4	12′-10	12	22	#4	12	13	#4	12	83.8
4′-8	4′-7	#8	6′-0	15	#4	13′-10	12	24	#4	12	14	#4	12	100.8
5′-0	4′-8	#8	6′-0	12	#4	14′-10	12	26	#4	12	15	#4	12	119.7
5′-5	5′-1	#9	6′-6	13	#4	15′-10	12	29	#4	12	16	#4	12	146.3
5′-11	5′-4	#9	6′-9	12	#4	16′-10	12	30	#4	12	17	#4	12	167.2
6′-1	5′-8	#10	7′-3	13	#4	17′-10	12	31	#4	12	18	#4	12	189.1
6′-6	5′-9	#10	7′-3	11	#4	18′-10	12	32	#4	12	19	#4	12	236.3
6′-11	5′-10	#10	7′-6	10	#4	19′-10	12	33	#4	12	20	#4	12	276.2

CANTILEVERED RETAINING WALLS—HIGHWAY SURCHARGE

Vertical steel in back of wall may be:

O + *T* only	*M* + *N* + 1Q
O only	*M* + Q + *N* + Q
M + *N* alt.	(as shown)

Comparison of bar spacing in table will indicate combination, spacings being accumulative from a selected *M* or O bar.



Wt of earth = 100 pcf

L of Internal Friction = ϕ = 33° 40′

f_s = 20,000 psi
f'_c = 3,000 psi
f_c = 1,350 psi
v = 90 psi
u = 300 psi

O & *T* bars alternate, or occur O + *T* + *T* except in walls without key, where only O bars are required.

REINFORCEMENT

Height of Wall = *h* (ft)	*M*			*N*			Q			*P*			O		
	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)
3	—	—	—	—	—	—	—	—	—	#3	1′-6	18	#3	4′-11	18
4	—	—	—	—	—	—	—	—	—	#3	1′-9	13	#3	6′-2	13
5	—	—	—	—	—	—	—	—	—	#3	2′-3	13	#3	7′-5	13
6	—	—	—	—	—	—	—	—	—	#4	2′-9	12	#3	8′-9	12
7	—	—	—	—	—	—	—	—	—	#4	3′-0	8	#4	10′-1	16
8	#4	7′-10	13	—	—	—	—	—	—	#4	3′-3	6½	#4	6′-0	13
9	#5	8′-10	16	—	—	—	—	—	—	#6	4′-0	16	#5	6′-8½	16
10	#6	9′-10	13	—	—	—	—	—	—	#6	4′-3	13	#5	7′-1	13
11	#5	10′-10	20	#6	5′-9	20	—	—	—	#6	4′-6	10	#5	6′-10	10
12	#4	11′-10	18	#4	8′-0	18	#4	6′-0	18	#7	5′-0	12	#6	8′-0	12
13	#4	12′-10	16½	#5	9′-0	16½	#5	6′-0	16½	#7	5′-3	11	#6	8′-3	11
14	#5	13′-10	19½	#6	9′-6	19½	#6	7′-0	19½	#8	5′-9	13	#7	9′-1	13
15	#6	14′-10	18	#6	10′-6	18	#6	7′-0	18	#9	6′-0	12	#7	8′-4	12
16	#6	15′-10	19½	#6	11′-0	19½	#6	8′-0	19½	#10	6′-6	13	#8	9′-11	13
17	#6	16′-10	16½	#6	12′-0	16½	#6	8′-6	16½	#10	7′-0	11	#8	10′-4	11
18	#6	17′-10	15	#6	13′-0	15	#7	9′-6	15	#9	7′-0	10	#8	10′-8	10
19	#7	18′-10	18	#8	14′-0	18	#8	11′-0	18	#11	7′-6	12	#9	11′-5	12
20	#7	19′-10	15	#8	15′-0	15	#8	11′-6	15	#10	7′-9	10	#9	10′-1	10

CANTILEVERED RETAINING WALLS—HIGHWAY SURCHARGE

CONCRETE OUTLINES

Height of Wall = h (ft)	B (ft)	a (ft)	b (ft)	c (ft)	Base Pressure at Toe (psf)	Base Pressure at Heel (psf)	Resisting Moment (lb-ft)	Overturning Moment (lb-ft)	$\frac{M_R}{M_O}$	Concrete (cf per lf of wall)
3	3'-0	0'-10	1'-1	1'-1	752	0	1,899	900	2.11	5.04
4	3'-5½	0'-10½	1'-3	1'-4	878	0	3,418	1,581	2.16	6.42
5	4'-1	0'-11	1'-5	1'-9	1092	0	5,186	2,501	2.07	8.07
6	4'-8	0'-11½	1'-7	2'-1½	1230	0	7,833	3,698	2.12	9.75
7	5'-2	1'-0	1'-9	2'-5	1384	0	10,967	5,194	2.11	11.41
8	5'-9	1'-1	2'-0	2'-8	1524	0	15,162	7,133	2.12	13.7
9	6'-3	1'-1½	2'-2	2'-11½	1683	0	19,820	9,372	2.11	15.7
10	6'-9½	1'-2	2'-4	3'-3½	1824	0	25,641	12,017	2.13	17.8
11	7'-3½	1'-2½	2'-6	3'-7	1974	0	32,183	15,121	2.13	19.8
12	7'-9	1'-3	2'-8	3'-10	2143	0	39,207	18,681	2.10	21.9
13	8'-4	1'-4	2'-10	4'-2	2281	0	48,979	22,890	2.14	24.9
14	8'-9½	1'-4½	3'-0	4'-5	2452	0	58,266	27,538	2.11	27.3
15	9'-4	1'-5	3'-2	4'-9	2591	0	69,950	32,765	2.13	29.8
16	9'-9½	1'-5½	3'-4	5'-0	2751	0	81,599	38,550	2.12	32.2
17	10'-4	1'-6	3'-6	5'-4	2880	0	96,094	45,000	2.14	34.9
18	10'-10	1'-7	3'-8	5'-7	3052	0	111,690	52,371	2.13	37.6
19	11'-3	1'-7½	3'-10	5'-9½	3232	0	126,583	60,228	2.10	41.4
20	11'-9	1'-8	4'-1	6'-0	3303	9	145,196	68,800	2.11	44.1

REINFORCEMENT

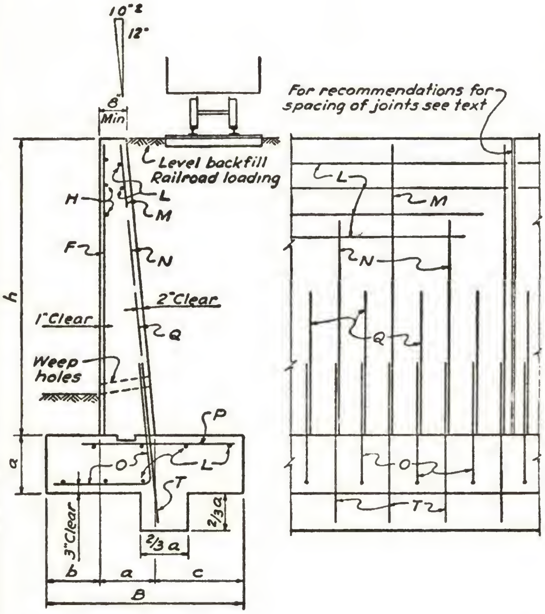
O a (ft)	O b (ft)	T Bar Size	T Length (ft)	T Spcg (in.)	F Bar Size	F Length (ft)	F Spcg (in.)	L Quant. of Bars	L Bar Size	L Spcg (in.)	H Quant. of Bars	H Bar Size	H Spcg (in.)	Weight of Bars (lb per lin. ft of wall)
1'-6	3'-5	—	—	—	#3	2'-10	18	7	#3	12	2	#3	18	5.45
1'-8½	4'-5½	—	—	—	#3	3'-10	18	8	#3	12	3	#3	18	7.46
1'-11	5'-6	#3	2'-6	13	#3	4'-10	18	10	#3	12	3	#3	18	10.30
2'-1½	6'-6½	#3	2'-9	12	#3	5'-10	18	12	#3	12	4	#3	18	13.13
2'-6	7'-7	#4	3'-3	16	#3	6'-10	12	13	#3	12	7	#3	12	19.77
2'-8	3'-4	#4	4'-3	13	#3	7'-10	12	15	#3	12	8	#3	12	26.8
2'-10½	3'-10	#5	4'-9	16	#3	8'-10	12	16	#3	12	9	#3	12	33.1
3'-1	4'-0	#5	4'-9	13	#4	9'-10	12	18	#4	12	10	#4	12	54.6
3'-3½	3'-6½	#5	4'-6	10	#4	10'-10	12	20	#4	12	11	#4	12	60.1
3'-6	4'-6	#6	5'-6	12	#4	11'-10	12	21	#4	12	12	#4	12	72.0
3'-9	4'-6	#6	5'-6	11	#4	12'-10	12	22	#4	12	13	#4	12	83.8
3'-11½	5'-1½	#7	6'-4	13	#4	13'-10	12	24	#4	12	14	#4	12	101.8
4'-2	4'-2	#7	5'-3	12	#4	14'-10	12	26	#4	12	15	#4	12	117.8
4'-4½	5'-6½	#8	7'-0	13	#4	15'-10	12	28	#4	12	16	#4	12	139.7
4'-7	5'-9	#8	8'-0	11	#4	16'-10	12	29	#4	12	17	#4	12	163.0
4'-10	5'-10	#8	7'-0	10	#4	17'-10	12	30	#4	12	18	#4	12	181.7
5'-0½	6'-4½	#9	7'-6	12	#4	18'-10	12	31	#4	12	19	#4	12	220.3
5'-4	4'-9	#9	7'-0	10	#4	19'-10	12	32	#4	12	20	#4	12	248.4

CANTILEVERED RETAINING WALLS—RAILWAY SURCHARGE

Vertical steel in back of wall may be:

$\left\{\begin{array}{ll} O + T \text{ only} & M + N + 1Q \\ O \text{ only} & M + Q + N + Q \\ M + N \text{ alt} & \text{(as shown)} \end{array}\right\}$

Comparison of bar spacings in table will indicate combination, spacings being accumulative from a selected *M* or O bar.



Wt. of earth = 100 pcf
L of Internal Friction = ϕ = 33° 40′
f_s = 20,000 psi
f'_c = 3,000 psi
f_c = 1,350 psi
v = 90 psi
u = 300 psi

O & *T* bars alternate or occur O + *T* + *T* except in walls without key, where only O bars are required.

REINFORCEMENT

Height of Wall = *h* (ft)	*M*			*N*			*Q*			*P*			O		
	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)	Bar Size	Length (ft)	Spcg (in.)
3	—	—	—	—	—	—	—	—	—	#7	3′-6	18	#3	5′-10	18
4	—	—	—	—	—	—	—	—	—	#7	3′-9	18	#3	7′-6	9
5	—	—	—	—	—	—	—	—	—	#7	3′-9	14	#4	9′-0	14
6	—	—	—	—	—	—	—	—	—	#8	4′-6	17	#5	10′-5	17
7	#5	6′-10	18	—	—	—	—	—	—	#9	4′-9	18	#6	7′-7	18
8	#5	7′-10	14	—	—	—	—	—	—	#9	5′-0	14	#6	7′-7	14
9	#4	8′-10	12	#4	5′-0	12	—	—	—	#8	5′-3	12	#6	8′-7	12
10	#4	9′-10	14	#4	5′-0	14	—	—	—	#9	5′-9	14	#7	9′-1	14
11	#5	10′-10	16	#5	5′-9	16	—	—	—	#10	6′-0	16	#8	9′-11	16
12	#5	11′-10	14	#6	6′-0	14	—	—	—	#10	6′-6	14	#8	10′-4	14
13	#5	12′-10	13	#6	6′-6	13	—	—	—	#10	6′-6	13	#8	10′-8	13
14	#5	13′-10	18	#6	10′-0	18	#6	6′-0	18	#10	7′-0	12	#8	11′-0	12
15	#6	14′-10	19½	#7	10′-9	19½	#7	6′-6	19½	#11	7′-9	13	#9	11′-5	13
16	#6	15′-10	18	#7	11′-3	18	#7	6′-9	18	#11	7′-9	12	#9	11′-8	12
17	#7	16′-10	18	#7	11′-3	18	#7	7′-0	18	#11	8′-0	12	#9	12′-0	12
18	#7	17′-10	16½	#7	11′-6	16½	#8	7′-0	16½	#9	8′-3	6½	#9	12′-5	11
19	#7	18′-10	15	#7	11′-6	15	#8	7′-0	15	#9	8′-6	5	#9	12′-10	10
20	#5	19′-10	18	#5	11′-9	18	#5	7′-6	9/18	#9	9′-0	4½	#9	13′-0	9

CANTILEVERED RETAINING WALLS—RAILWAY SURCHARGE

CONCRETE OUTLINES

Height of Wall = h (ft)	B (ft)	a (ft)	b (ft)	c (ft)	Base Pressure at Toe (psf)	Base Pressure at Heel (psf)	Resisting Moment (lb-ft)	Overturning Moment (lb-ft)	$\frac{M_R}{M_O}$	Concrete (cf per lf of wall)
3	5′-1	0′-11	1′-10	2′-4	735	0	5,413	2,492	2.17	7.41
4	5′-11	1′-0	2′-4	2′-7	852	0	8,979	4,180	2.15	9.68
5	6′-7	1′-1	2′-8	2′-10	1,018	0	13,226	6,367	2.08	11.98
6	7′-4	1′-2	2′-11	3′-3	1,171	0	19,263	9,140	2.11	14.71
7	8′-0	1′-3	3′-3	3′-6	1,313	0	25,957	12,432	2.09	17.41
8	8′-8	1′-4	3′-7	3′-9	1,451	0	33,980	16,371	2.08	20.3
9	9′-4	1′-5	3′-11	4′-0	1,577	0	43,714	20,984	2.08	23.5
10	10′-0	1′-6	4′-3	4′-3	1,699	9	54,795	26,235	2.09	26.8
11	10′-6	1′-7	4′-5	4′-6	1,890	0	66,050	32,222	2.05	30.1
12	11′-2	1′-8	4′-8	4′-10	2,021	0	81,107	39,006	2.08	33.8
13	11′-9	1′-9	4′-10	5′-2	2,188	0	96,976	46,519	2.08	37.7
14	12′-4	1′-10	5′-0	5′-6	2,343	12	114,726	54,888	2.09	41.6
15	12′-11	1′-11	5′-3	5′-9	2,493	13	134,313	64,193	2.09	45.8
16	13′-6	2′-0	5′-4	6′-2	2,670	24	156,851	74,309	2.11	50.1
17	14′-1	2′-1	5′-6	6′-6	2,833	29	180,983	85,429	2.12	54.7
18	14′-9	2′-2	5′-8	6′-11	2,974	76	210,560	97,549	2.16	59.7
19	15′-3	2′-3	5′-10	7′-2	3,146	58	237,054	110,598	2.14	64.3
20	15′-10	2′-4	6′-0	7′-6	3,296	76	268,602	124,688	2.15	69.3

REINFORCEMENT

O a (ft)	b (ft)	T Bar Size	T Length (ft)	T Spcg (in.)	F Bar Size	F Length (ft)	F Spcg (in.)	L Quant. of Bars	L Bar Size	L Spcg (in.)	H Quant. of Bars	H Bar Size	H Spcg (in.)	Weight of Bars (lb per lin. ft of wall)
2′-4	3′-6	—	—	—	#3	2′-10	18	8	#3	12	2	#3	18	10.44
2′-11	4′-7	#3	3′-0	9	#3	3′-10	18	9	#3	12	3	#3	18	15.22
3′-4	5′-8	#4	3′-6	14	#3	4′-10	18	10	#3	12	3	#3	18	19.44
3′-8	6′-9	#5	3′-9	17	#3	5′-10	18	12	#3	12	4	#3	18	25.79
4′-1	3′-6	#6	3′-9	18	#3	6′-10	18	14	#3	12	7	#3	12	36.5
4′-6	3′-1	#6	4′-0	14	#3	7′-10	12	16	#3	12	8	#3	12	48.3
4′-11	3′-8	#6	4′-6	12	#3	8′-10	12	18	#3	12	9	#3	12	56.4
5′-4	3′-9	#7	4′-9	14	#4	9′-10	12	20	#4	12	10	#4	12	75.9
5′-7	4′-4	#8	5′-3	16	#4	10′-10	12	22	#4	12	11	#4	12	91.9
5′-11	4′-5	#8	5′-6	14	#4	11′-10	12	24	#4	12	12	#4	12	110.8
6′-2	4′-6	#8	5′-9	13	#4	12′-10	12	26	#4	12	13	#4	12	122.1
6′-5	4′-7	#8	6′-0	12	#4	13′-10	12	28	#4	12	14	#4	12	139.1
6′-9	4′-8	#9	6′-3	13	#4	14′-10	12	30	#4	12	15	#4	12	168.5
6′-11	4′-9	#9	6′-3	12	#4	15′-10	12	32	#4	12	16	#4	12	185.1
7′-2	4′-10	#9	6′-3	12	#4	16′-10	12	33	#4	12	17	#4	12	200.3
7′-5	5′-0	#9	6′-6	11	#4	17′-10	12	35	#4	12	18	#4	12	226.0
7′-10	5′-0	#9	6′-6	10	#4	18′-10	12	36	#4	12	19	#4	12	261.9
7′-11	5′-1	#9	6′-9	9	#4	19′-10	12	37	#4	12	20	#4	12	296.4

SPECIAL ELL-SHAPED WALLS

Other types of retaining wall are frequently used, the most common being an ell shape, used where the face of the stem is exactly on a property line and the base can not project beyond this point. Two cases of ell-shaped walls are common:—(1) where the heel projects back under the earth which is at the high level, (2) where the heel projects under the earth which is at the low level. In the first case, a fair approximation can be made by shifting the position of the stem to the front of the toe for any of the four cases illustrated on pages 384 to 391. In the latter case, the base of the wall would have to be increased by 33⅓ to 50 per cent to obtain stability. The number of conditions of special walls is so great that it is impracticable to tabulate them all, but the above modifications to the tables, coupled with the earth pressure tables on pages 394 and 395 and the outline of a wall design illustrated in the example on page 380 ff, should enable the user to satisfy his needs.

BASEMENT WALLS SPANNED VERTICALLY

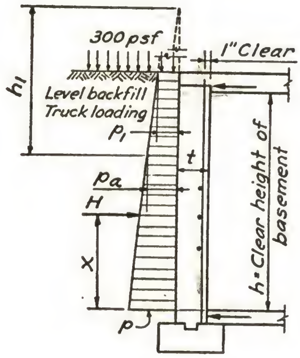
Another common case of walls resisting earth pressure occurs in basement walls spanned vertically from basement floor slab to first floor slab. The table on page 393 suggests suitable reinforcement for walls of 8-, 10-, 12- and 15-inch thickness on story heights from 7 to 18 feet, all based upon a surcharge of 300 psf to represent ordinary working loads on top of the backfill. Should neighboring buildings, adjoining railroad tracks, or other conditions impose a heavier surcharge, the wall reinforcement and thickness should be increased accordingly.

A table is also given for the maximum vertical heights of unreinforced basement walls of the same thicknesses to keep the tension developed by bending within 100 psi both for 300 psf surcharge and for no surcharge at all.

In all cases of vertically spanned walls, a word of caution on the design drawings is advisable to see that the contractor installs both the basement floor slab and the first floor slab and has them able to receive thrust before backfilling against the basement wall. Of course, both slabs should bear against the wall to afford horizontal support.

RETAINING WALLS

BASEMENT WALLS SPANNED VERTICALLY



Angle of Repose — $\phi = 33°40'$

$$p_a = wh_1\frac{1 - \sin\phi}{1 + \sin\phi} \qquad H = \frac{p + p_1}{2}h$$

$$w = 100 \qquad x = \frac{h}{3}\left(\frac{p + 2p_1}{p + p_1}\right)$$

STRESSES:—

f_s = 20,000 psi
f'_c = 3,000 psi
f_c = 1,350 psi
v_c = 90 psi
v = 300 psi

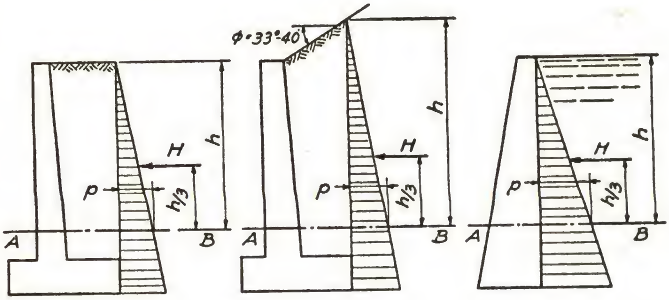
Wall Thickness (t)	8"		10"		12"		15"	
Clear Height of Basement (h)	Reinforcement on Interior Side of Wall							
	Vert.	Horiz.	Vert.	Horiz.	Vert.	Horiz.	Vert.	Horiz.
7'-0	#4 @ 14"	#4 @ 10"	#4 @ 13"	#5 @ 12"	#4 @ 11"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
8'-0	#4 @ 12"	#4 @ 10"	#4 @ 13"	#5 @ 12"	#4 @ 11"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
9'-0	#4 @ 11"	#4 @ 10"	#4 @ 13"	#5 @ 12"	#4 @ 11"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
10'-0	#5 @ 13"	#4 @ 10"	#4 @ 10½"	#5 @ 12"	#4 @ 11"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
11'-0	#6 @ 14"	#4 @ 10"	#5 @ 13"	#5 @ 12"	#4 @ 10"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
12'-0	#6 @ 11½"	#4 @ 10"	#5 @ 10½"	#5 @ 12"	#5 @ 12"	#5 @ 10"	#5 @ 13½"	#6 @ 12"
13'-0	#7 @ 12"	#4 @ 10"	#6 @ 12"	#5 @ 12"	#5 @ 10"	#5 @ 10"	#5 @ 12"	#6 @ 12"
14'-0	#8 @ 12"	#4 @ 10"	#7 @ 12½"	#5 @ 12"	#6 @ 11"	#5 @ 10"	#5 @ 10"	#6 @ 12"
15'-0			#8 @ 13½"	#5 @ 12"	#7 @ 12½"	#5 @ 10"	#6 @ 12"	#6 @ 12"
16'-0			#9 @ 13"	#5 @ 12"	#8 @ 14"	#5 @ 10"	#7 @ 13½"	#6 @ 12"
17'-0			#9 @ 12"	#5 @ 12"	#8 @ 13"	#5 @ 10"	#7 @ 11½"	#6 @ 12"
18'-0							#8 @ 13"	#6 @ 12"

Often a pair of bars is added at top and at bottom of a wall to provide longitudinal beam action from footing to footing, pile cap to pile cap, or to bridge across soft and firm subsoil conditions.

MAXIMUM HEIGHTS IN FEET OF UNREINFORCED BASEMENT WALLS (KEEPING TENSION $\lessapprox$ 100 PSI).

Wall Thickness (t)	Surcharge = 300 psf (Diagram above)	No Surcharge
8"	6.9	8.2
10"	8.0	9.7
12"	9.4	11.0
15"	11.1	12.7

RETAINING WALLS
EARTH AND WATER PRESSURES



p—Intensity of horizontal pressure (psf) at any depth (h).

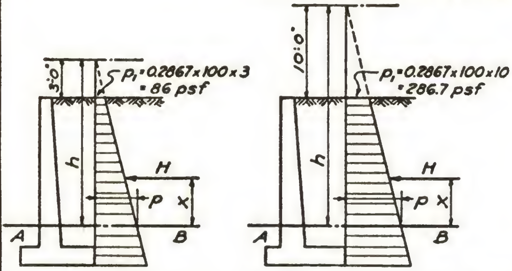
H—Total horizontal pressure (lb) above A-B.

M—Moment (lb-in.) of H about an axis lying in Section A-B.

ϕ—Angle of repose = 33°40′ (1 on 1½). Rankine Theory.

Depth "h"	Earth Horizontal (w = 100 pcf)			Earth Sloping Upward at ϕ = 33° 40′ (w = 100 pcf)			Water (w = 62.5 pcf)		
	p = 0.2867 wh	$H = \frac{ph}{2}$ = 0.1434 wh^2	$M = \frac{Hh}{3} 12$ = 0.5734 wh^3	p = 0.6927 wh	$H = \frac{ph}{2}$ = 0.3463 wh^2	$M = \frac{Hh}{3} 12$ = 1.3853 wh^3	$p = wh$	$H = \frac{wh^2}{2}$	$M = 2wh^3$
ft	psf	lb	lb-in.	psf	lb	lb-in.	psf	lb	lb-in.
1	29	14	69	89	35	138	63	31	125
2	57	57	459	139	139	1,108	125	125	1,000
3	86	129	1,548	208	312	3,740	188	281	3,375
4	115	229	3,670	277	554	8,866	250	500	8,000
5	143	358	7,167	346	866	17,310	313	781	15,620
6	172	516	12,380	416	1,247	29,920	375	1,125	27,000
7	201	702	19,660	485	1,697	47,510	438	1,531	42,870
8	229	917	29,350	554	2,216	70,920	500	2,000	64,000
9	258	1,161	41,800	623	2,805	100,900	563	2,531	91,120
10	287	1,433	57,340	693	3,463	138,500	625	3,125	125,000
11	315	1,735	76,320	762	4,190	184,300	688	3,781	166,300
12	344	2,064	99,080	831	4,987	239,300	750	4,500	216,000
13	373	2,423	125,900	901	5,852	304,300	813	5,281	274,600
14	401	2,810	157,300	970	6,787	380,100	875	6,125	343,000
15	430	3,225	193,500	1039	7,792	467,500	938	7,031	421,800
16	459	3,670	234,800	1108	8,865	567,400	1000	8,000	512,000
17	487	4,143	281,700	1178	10,000	680,500	1063	9,031	614,100
18	516	4,645	334,400	1247	11,220	807,900	1125	10,120	729,000
19	545	5,175	393,200	1316	12,500	950,100	1188	11,280	857,300
20	573	5,734	458,700	1385	13,850	1,108,000	1250	12,500	1,000,000
21	602	6,322	531,000	1455	15,270	1,282,000	1313	13,780	1,157,000
22	631	6,938	610,500	1524	16,760	1,475,000	1375	15,120	1,331,000
23	659	7,583	697,600	1593	18,310	1,685,000	1438	16,530	1,520,000
24	688	8,257	792,600	1662	19,940	1,915,000	1500	18,000	1,728,000
25	717	8,959	895,900	1732	21,640	2,164,000	1563	19,530	1,953,000
26	745	9,690	1,007,000	1801	23,410	2,434,000	1625	21,120	2,197,000
27	774	10,450	1,128,000	1870	25,245	2,726,000	1688	22,780	2,460,000
28	803	11,230	1,258,000	1940	27,150	3,041,000	1750	24,500	2,744,000
29	831	12,050	1,398,000	2009	29,120	3,378,000	1813	26,280	3,048,000
30	860	12,900	1,548,000	2078	31,160	3,740,000	1875	28,120	3,375,000
31	889	13,770	1,708,000	2147	33,270	4,126,000	1938	30,030	3,723,000
32	917	14,670	1,878,000	2217	35,460	4,539,000	2000	32,000	4,096,000
33	946	15,610	2,060,000	2286	37,710	4,978,000	2063	34,030	4,492,000
34	975	16,570	2,253,000	2355	40,030	5,444,000	2125	36,120	4,913,000
35	1003	17,560	2,458,000	2424	42,420	5,939,000	2188	38,280	5,359,000

RETAINING WALLS
EARTH PRESSURE WITH SURCHARGE



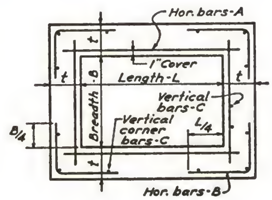
p—Intensity of horizontal pressure (psf) at any depth (h − 3) or (h − 10).
H—Total horizontal pressure (lb) above A-B.
M—Moment (lb-in.) of *H* about an axis lying in Section A-B.
ϕ—Angle of repose = 33°40′ (1 on 1½).
w—Weight of earth = 100 pcf.
Rankine Theory.

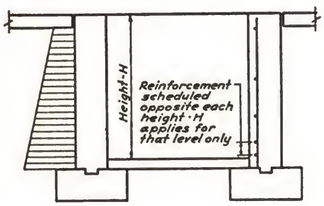
Depth of AB below surface	Horizontal Surcharge—3′-0″ High = 300 psf				Horizontal Surcharge—10′-0″ High = 1000 psf			
	Intensity of Horizontal Pressure at A-B	Total Horizontal Pressure above A-B	Arm of Total Horiz. Pressure	Moment of H about Sect. A-B	Intensity of Horizontal Pressure at A-B	Total Horizontal Pressure above A-B	Arm of Total Horiz. Pressure	Moment of H about Sect. A-B
	p	$H = \frac{(h-3)}{2}(p+p_1)$	$x = \frac{(h-3)(p+2p_1)}{3(p+p_1)}$	$M = 12\,Hx$	p	$H = \frac{(h-10)}{2}(p+p_1)$	$x = \frac{(h-10)(p+2p_1)}{3(p+p_1)}$	$M = 12\,Hx$
ft	psf	lb	ft	lb-in.	psf	lb	ft	lb-in.
1	115	100	0.476	574	315	301	0.492	1,777
2	143	229	0.917	2,520	343	630	0.970	7,333
3	172	387	1.333	6,190	372	989	1.435	17,031
4	201	574	1.732	11,930	401	1,376	1.889	31,191
5	229	787	2.121	20,031	429	1,791	2.334	50,162
6	258	1,032	2.500	30,960	458	2,236	2.769	74,298
7	287	1,305	2.871	44,960	487	2,710	3.197	103,966
8	315	1,604	3.238	62,325	515	3,209	3.619	139,360
9	344	1,935	3.600	83,592	544	3,741	4.034	181,100
10	373	2,295	3.957	109,000	573	4,302	4.444	229,400
11	401	2,678	4.314	138,700	601	4,886	4.850	284,400
12	430	3,096	4.666	173,400	630	5,504	5.250	346,800
13	459	3,542	5.017	213,300	659	6,151	5.646	416,800
14	487	4,011	5.367	258,300	687	6,820	6.040	494,400
15	516	4,515	5.714	309,600	716	7,525	6.429	580,600
16	545	5,048	6.060	367,100	745	8,259	6.814	675,400
17	573	5,601	6.406	430,600	773	9,013	7.199	778,700
18	602	6,192	6.750	501,600	802	9,804	7.579	891,700
19	631	6,811	7.092	579,700	831	10,620	7.957	1,015,000
20	659	7,450	7.436	664,800	859	11,460	8.340	1,148,000
21	688	8,127	7.777	758,500	888	12,340	8.700	1,290,000
22	717	8,833	8.118	860,500	917	13,240	9.070	1,441,000
23	745	9,556	8.460	970,200	945	14,170	9.450	1,608,000
24	774	10,320	8.800	1,090,000	974	15,130	9.810	1,781,000
25	803	11,110	9.139	1,219,000	1003	16,130	10.180	1,972,000
26	831	11,920	9.479	1,356,000	1031	17,130	10.550	2,170,000
27	860	12,770	9.818	1,505,000	1060	18,180	10.910	2,380,000
28	889	13,650	10.150	1,663,000	1089	19,260	11.270	2,605,000
29	917	14,540	10.490	1,830,000	1117	20,360	11.640	2,845,000
30	946	15,480	10.830	2,012,000	1146	21,500	12.000	3,097,000
31	975	16,440	11.170	2,204,000	1175	22,660	12.350	3,358,000
32	1003	17,420	11.500	2,404,000	1203	23,840	12.710	3,636,000
33	1032	18,440	11.840	2,620,000	1232	25,070	13.070	3,932,000
34	1061	19,500	12.180	2,850,000	1261	26,320	13.430	4,243,000
35	1089	20,560	12.520	3,089,000	1289	27,580	13.780	4,561,000

WALLS FOR PITS

Pits are frequent in industrial and commercial structures. There are several ways of designing the walls to resist the lateral pressure of the earth. Walls tabulated below were designed for a surcharge of 300 psf, $f_s = 20{,}000$ psi, $f'_c = 3{,}000$ psi, $n = 10$, ($p = 28.6$ psf).

If there is any possibility of seepage or spillage, pit floors should be sloped to a suitable drain or sump (with grease or oil trap if conditions require). Pits should have ladder rungs for easy access. Under wet soil conditions, pit walls should be waterproofed with exterior membrane, ironiting, or integral waterproofing, to suit degree of exposure, with a continuous waterstop in construction joints. Floor slabs around pits often rest in a recess around top of wall to maintain floors flush with top of pit. Provide inserts for any pipe railings, curb angles, floor beams, or gratings.





Case I—Rectangular, relatively deep, open-top pits of moderate length and width. For rectangular, moderately deep pits, especially when the bottom slab is not structurally integral, it is often economical to span the side walls as slabs from end wall to end wall, and the end walls from side wall to side wall, reinforcing around the corners to develop negative moments.

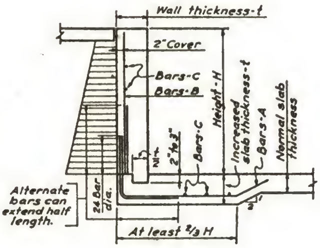
When $B \geq \frac{3}{4} L$, use distance L and same inside bars for all four walls; when $B < \frac{3L}{4}$, design as quadrangular frame.

Bars B are to be spaced same as Bars A and one size larger, viz., #5 for #4, and #6 for #5.

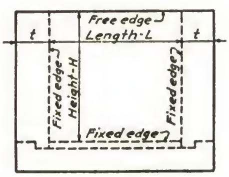
Height H	Bars A for Length L											Wall Thickness and Bars C
	10'-0	11'-0	12'-0	13'-0	14'-0	15'-0	16'-0	17'-0	18'-0	19'-0	20'-0	
7'-0	#4@12	#4@12	#4@12	#4@12	#4@12	#4@12	#4@12	#4@11	#4@10	#4@9	#4@8	t = 8" C = #4@12
8'-0	#4@12	#4@12	#4@12	#4@11	#4@10	#4@9	#4@9	#4@8	#4@8	#5@12	#5@11	
9'-0	#4@12	#4@12	#4@11	#4@10	#4@9	#4@8	#5@12	#5@11	#5@10	#5@9	#5@9	
10'-0	#4@11	#4@10	#4@9	#4@8	#4@8	#5@11	#5@10	#5@9	#5@8	#5@8	#5@11	t = 10" C = #4@10
11'-0	#4@10	#4@9	#4@8	#5@11	#5@10	#5@9	#5@8	#5@8	#5@10	#5@9	#5@9	
12'-0	#4@9	#4@8	#5@11	#5@10	#5@9	#5@8	#5@8	#5@10	#5@9	#5@8	#5@8	
13'-0	#4@8	#5@11	#5@10	#5@9	#5@8	#5@9	#5@10	#5@9	#5@8	#5@10	#5@9	t = 12" C = #5@12
14'-0	#5@11	#5@10	#5@10	#5@8	#5@10	#5@9	#5@8	#5@8	#5@9	#5@9	#5@8	
15'-0	#5@10	#5@9	#5@8	#5@10	#5@9	#5@8	#5@8	#5@10	#5@8	#5@8	#6@10	

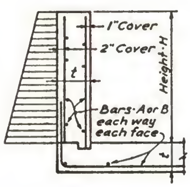
WALLS FOR PITS

Case II—Long, relatively shallow, open-top pits. For pits over, say, 20 ft on a side or with broken sides so that end walls can not lean against side walls, the simplest wall design is one vertically cantilevered from the floor slab. Wall thicknesses and reinforcement for various heights are scheduled.



H	t	BARS A	BARS B	BARS C
0 to 4'-0	8"	#4 @ 12	#4 @ 12	#4 @ 12
5'-0	8"	#5 @ 12	#4 @ 12	#4 @ 12
6'-0	8"	#5 @ 10	#4 @ 12	#4 @ 12
7'-0	8"	#6 @ 10	#4 @ 12	#4 @ 12
8'-0	10"	#6 @ 10	#5 @ 12	#5 @ 12
9'-0	10"	#7 @ 11	#5 @ 12	#5 @ 12
10'-0	10"	#7 @ 9	#5 @ 12	#5 @ 12





Case III—Cases I and II may be combined into a wall fixed on three sides, free at the top, undergoing trapezoidal earth pressure, which, of course, is somewhat more economical utilization of the reinforcement.

Height H	Bars A or B, Each Way, Each Face, in Length L: 10'-0	11'-0	12'-0	13'-0	14'-0	15'-0	16'-0	17'-0	18'-0	19'-0	20'-0	Wall Thickness t
7'-0	#4@12	#4@12	#4@12	#4@12	#4@12	#4@12	#5@12	#5@12	#5@12	#5@12	#5@12	8"
8'-0	#4@12	#4@12	#4@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	10"
9'-0	#4@12	#4@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	
10'-0	#4@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	12"
11'-0	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	
12'-0	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	15"
13'-0	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	
14'-0	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12		
15'-0	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12	#5@12			

Case IV—Walls for pits with top slabs may be treated as basement walls. See page 393.

Tentative Specifications for
BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT[1]

ASTM Designation: A 15–54 T

ISSUED, 1950; Revised, 1952, June and October 1954.[2]

These Tentative Specifications have been approved by the sponsoring committee and accepted by the Society in accordance with established procedures, for use pending adoption as standard. Suggestions for revisions should be addressed to the Society at 1916 Race St., Philadelphia 3, Pa.

Scope

1. (*a*) These specifications cover two classes of billet-steel concrete reinforcement bars: namely, plain and deformed. A deformed bar is defined as a bar which conforms to the latest issue of the Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (ASTM Designation: A 305).[3] The standard sizes of deformed bars with number designations shall be those listed in Table I. The standard sizes of plain bars shall be designated by their nominal diameters.

TABLE I.—DEFORMED BAR DESIGNATION NUMBERS, UNIT WEIGHTS, AND NOMINAL DIMENSIONS.

Bar Designation Number [a]	Unit Weight, lb. per ft.	Nominal Dimensions		
		Diameter, in.	Cross-Sectional Area, sq. in.	Perimeter, in.
[b].......	0.167	0.250	0.05	0.786
3........	0.376	0.375	0.11	1.178
4........	0.668	0.500	0.20	1.571
5........	1.043	0.625	0.31	1.963
6........	1.502	0.750	0.44	2.356
7........	2.044	0.875	0.60	2.749
8........	2.670	1.000	0.79	3.142
9 [c].......	3.400	1.128	1.00	3.544
10 [c].......	4.303	1.270	1.27	3.990
11 [c].......	5.313	1.410	1.56	4.430

[a] Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.

[b] ¼-in. bar in plain round only.

[c] Bars of designation Nos. 9, 10, and 11 correspond to the former 1-in. square, 1⅛-in. square, and 1¼-in. square sizes and are equivalent to those former standard bar sizes in weight and nominal cross-sectional areas.

NOTE.—The above table including the footnotes is in agreement with U. S. Department of Commerce Simplified Practice Recommendation 26-50 covering Steel Reinforcing Bars.

[1] Under the standardization procedure of the Society, these specifications are under the jurisdiction of the ASTM Committee A-1 on Steel.

[2] Latest revisions accepted by the Society at the Annual Meeting, June, 1954, and by the Administrative Committee on Standards, October 4, 1954.

Prior to their publication as tentative, these specifications were published as standard from 1911 to 1950, being revised in 1912, 1913, 1914, 1930, 1933, 1935, 1939 and 1950.

[3] See page 408.

BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT

(*b*) Plain and deformed bars are of three grades: namely, structural, intermediate, and hard.

Process

2. (*a*) The steel shall be made by one or more of the following processes: open-hearth, electric-furnace, or acid-bessemer.

(*b*) The bars shall be rolled from billets directly reduced from ingots of properly identified heats of open-hearth or electric-furnace steel, or lots of acid-bessemer steel.

Chemical Composition

3. The steel shall conform to the following requirements as to chemical composition:

Phosphorus, max., per cent:		
Open-hearth or electric-furnace	Basic	0.05
	Acid	0.08
Acid-bessemer and open-hearth or electric-furnace rephosphorized		0.12

Ladle Analysis

4. (*a*) An analysis of each heat of open-hearth or electric-furnace steel shall be made to determine the percentages of carbon, manganese, phosphorus, and sulfur.

(*b*) Carbon and manganese determinations shall be made of each blow of bessemer steel, and determinations for phosphorus and sulfur representing the average of the blows applied for each 8-hr. period.

(*c*) The analyses prescribed in Paragraphs (*a*) and (*b*) shall be made by the manufacturer from test ingots taken during the pouring of the heats or blows. The chemical composition thus determined shall be reported to the purchaser or his representative, and the percentage of phosphorus shall conform to the requirements specified in Section 3.

Check Analysis

5. An analysis may be made by the purchaser from finished bars representing each heat of open-hearth or electric-furnace steel, and each blow or lot of ten tons of bessemer steel. The phosphorus content thus determined shall not exceed that specified in Section 3 by more than 25 per cent.

Tensile Properties

6. (*a*) The material shall conform to the requirements as to tensile properties prescribed in Table II.

(*b*) The yield point shall be determined by the drop of the beam or halt in the gage of the testing machine.

(*c*) For plain bars over ¾ in. in diameter, a deduction from the percentages of elongation prescribed in Table II of 0.25 per cent shall be made for each increase of 1/32 in. of the specified diameter above ¾ in.

(*d*) For deformed bars over No. 6 bar (nominal diameter ¾ in.) a deduction from the percentages of elongation prescribed in Table II of 1.00 per cent shall be made for each increase in bar number.

BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT

TABLE II.—TENSILE REQUIREMENTS.

	Plain Bars			Deformed Bars		
	Structural Grade	Intermediate Grade	Hard Grade	Structural Grade	Intermediate Grade	Hard Grade
Tensile strength, psi	55 000 to 75 000	70 000 to 90 000	80 000 min.	55 000 to 75 000	70 000 to 90 000	80 000 min.
Yield point, min., psi	33 000	40 000	50 000	33 000	40 000	50 000
Elongation in 8 in., min., per cent	1 400 000 [a] / tens. str. but not less than 20 per cent [a]	1 300 000 [a] / tens. str. but not less than 16 per cent [a]	1 100 000 [a] / tens. str.	1 200 000 [b] / tens. str. but not less than 16 per cent [b]	1 100 000 [b] / tens. str. but not less than 12 per cent [b]	1 000 000 [b] / tens. str.

[a] See Section 6 (*c*) and (*e*).
[b] See Section 6 (*d*) and (*f*).

(*e*) For plain bars under 7/16 in. in diameter, a deduction from the percentages of elongation prescribed in Table II of 0.5 per cent shall be made for each decrease of 1/32 in. of the specified diameter below 7/16 in.

(*f*) For No. 3 deformed bar (nominal diameter 3/8 in.) a deduction from the percentage of elongation prescribed in Table II of 1.00 per cent shall be made.

TABLE III.—BEND TEST REQUIREMENTS FOR PLAIN BARS.[a]

Diameter of Bar, in.	Structural Grade	Intermediate Grade	Hard Grade
Under 3/4	180 deg. $d = t$	180 deg. $d = 2t$	180 deg. $d = 4t$
3/4 and over	180 deg. $d = t$	90 deg. $d = 2t$	90 deg. $d = 4t$

[a] On plain bars whose application is in unbent form such as load transfer dowels, the cold bend test shall be waived.

NOTE.—d = diameter of pin around which specimen is bent, and
t = diameter of the specimen.

Bending Properties

7. (*a*) The bend test specimen shall stand being bent, at room temperature, around a pin without cracking on the outside of the bent portion. The requirements for degree of bending and sizes of pins prescribed in Table III for plain bars or Table IV for deformed bars shall be observed.

(*b*) The bend test shall be made on specimens of sufficient length to insure free bending and with apparatus which provides:

(*1*) Continuous and uniform application of force throughout the duration of the bending operation,

(2) Unrestricted movement of the specimen at points of contact with the apparatus, and

(3) Close wrapping of the specimen around the pin or mandrel during the bending operation.

(c) Other methods of bend testing may be used, but failures due to such methods shall not constitute a basis for rejection.

TABLE IV.—BEND TEST REQUIREMENTS FOR DEFORMED BARS.

Bar Designation Number	Structural Grade	Intermediate Grade	Hard Grade
Under No. 6	180 deg. $d = 2t$	90 deg. $d = 3t$	90 deg. $d = 4t$
Nos. 6, 7, 8	180 deg. $d = 3t$	90 deg. $d = 4t$	90 deg. $d = 5t$
Nos. 9, 10, 11	180 deg. $d = 4t$	90 deg. $d = 5t$	90 deg. $d = 6t$

NOTE.—d = diameter of pin around which the specimen is bent, and t = diameter of the specimen.

Test Specimens

8. Tension and bend test specimens from plain or deformed bars shall be of the full section of bars as rolled. For tension tests of deformed bars the sectional area used for unit stress determinations shall be calculated from the length and weight of the test specimen (Note).

NOTE.—The area in square inches may be calculated by dividing the weight per linear inch of specimen in pounds by 0.2833 (weight of 1 cu. in. of steel), or by dividing the weight per linear foot of specimen in pounds by 3.4 (weight of steel 1 in. square and 1 ft. long).

Number of Tests

9. (a) One tension test and one bend test shall be made from each heat of open-hearth or electric-furnace steel, and from each blow or lot of ten tons of bessemer steel. If, however, material from one heat or blow differs ⅜ in. or more in diameter in the case of plain bars, or by three or more designation numbers in the case of deformed bars, one tension and one bend test shall be made from both the largest and smallest plain bars, and from the highest and lowest designation number of the deformed bars rolled.

(b) If any test specimen develops flaws, it may be discarded and another specimen substituted.

(c) If the percentage of elongation of any tension test specimen is less than that specified in Section 6 and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

Permissible Variations in Weight

10. The permissible variations in weight shall not exceed the limits prescribed in Table V.

Finish

11. The bars shall be free from injurious defects and shall have a workmanlike finish.

BILLET-STEEL BARS FOR CONCRETE REINFORCEMENT

TABLE V.—PERMISSIBLE VARIATIONS FROM THEORETICAL WEIGHTS.

NOTE.—The theoretical weights for deformed bars listed in Table I and the established standard weights for plain bars rolled to fractions of inches shall be used to establish conformance to this table.

Diameter of Bars, in.	Permissible Variations from Theoretical Weights	
	Lot,[a] Over or Under, per cent	Individual Bar, Under, per cent
Plain bars:		
Under 3/8	5	10
3/8 and over	3.5	6
Deformed bars, all sizes	3.5	6

[a] The term "lot" means all the bars of the same nominal weight per linear foot in a carload.

Marking

12. The brand of the manufacturer shall be legibly rolled on all deformed bars. For the purpose of identification, a distinctive pattern is considered to be a manufacturer's brand. When loaded for mill shipment, all bars shall be properly separated and tagged with the manufacturer's test identification number.

Inspection

13. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works that concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests (except check analysis) and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection

14. (*a*) Unless otherwise specified, any rejection based on tests made in accordance with Section 5 shall be reported to the manufacturer within five working days from the receipt of samples by the purchaser.

(*b*) Material that shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Rehearing

15. Samples tested in accordance with Section 5 that represent rejected material shall be preserved for two weeks from the date rejection is reported to the manufacturer. In case of dissatisfaction with the results of the tests, the manufacturer may make claim for a rehearing within that time.

Tentative Specifications for

RAIL-STEEL BARS FOR CONCRETE REINFORCEMENT[1]

ASTM Designation: A 16–54 T

ISSUED, 1950; Revised, 1952, 1954.[2]

These Tentative Specifications have been approved by the sponsoring committee and accepted by the Society in accordance with established procedures, for use pending adoption as standard. Suggestions for revisions should be addressed to the Society at 1916 Race St., Philadelphia 3, Pa.

Scope

1. These specifications cover two classes of rail-steel concrete reinforcement bars: namely, plain and deformed. A deformed bar is defined as a bar which conforms to the latest issue of the Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (ASTM Designation: A 305).[3] The standard sizes of deformed bars with number designations shall be those listed in Table I. The standard sizes of plain bars shall be designated by their nominal diameters.

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3....	0.376	0.375	0.11	1.178
4....	0.668	0.500	0.20	1.571
5....	1.043	0.625	0.31	1.963
6....	1.502	0.750	0.44	2.356
7....	2.044	0.875	0.60	2.749
8....	2.670	1.000	0.79	3.142
9[c]...	3.400	1.128	1.00	3.544
10[c]...	4.303	1.270	1.27	3.990
11[c]...	5.313	1.410	1.56	4.430

[a] Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.

[b] ¼-in. bar in plain round only.

[c] Bars of designation Nos. 9, 10, and 11 correspond to the former 1-in. square, 1⅛-in. square, and 1¼-in. square sizes and are equivalent to those former standard bar sizes in weight and nominal cross-sectional areas.

NOTE.—The above table including the footnotes is in agreement with U. S. Department of Commerce Simplified Practice Recommendation 26-50 covering Steel Reinforcing Bars.

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[1] Under the standardization procedure of the Society, these specifications are under the jurisdiction of the ASTM Committee A-1 on Steel.

[2] Latest revision accepted by the Society at the Annual Meeting, June, 1954.

Prior to their publication as tentative, these specifications were published as standard from 1913 to 1950, being revised in 1914, 1933, 1935, and 1950.

[3] See page 408.

RAIL-STEEL BARS FOR CONCRETE REINFORCEMENT

Manufacture

2. The bars shall be rolled from standard section Tee rails. No other materials such as those known by the terms "rerolled," "rail-steel equivalent," and "rail-steel quality" shall be substituted.

Tensile Properties

3. (*a*) The material shall conform to the requirements as to tensile properties prescribed in Table II.

TABLE II.—TENSILE REQUIREMENTS.

	Plain Bars	Deformed Bars
Tensile strength, min., psi..........	80 000	80 000
Yield point, min., psi..............	50 000	50 000
Elongation in 8 in., min., per cent...	1 100 000 [a]	1 000 000 [b]
	tens. str.	tens. str.

[a] See Section 3 (*c*) and (*e*).
[b] See Section 3 (*d*) and (*f*).

(*b*) The yield point shall be determined by the drop of the beam or halt in the gage of the testing machine.

(*c*) For plain bars over ¾ in. in diameter, a deduction from the percentages of elongation prescribed in Table II of 0.25 per cent shall be made for each increase of 1/32 in. of the specified diameter above ¾ in.

(*d*) For deformed bars over No. 6 bar (nominal diameter ¾ in.) a deduction from the percentages of elongation prescribed in Table II of 1.00 per cent shall be made for each increase in bar number.

(*e*) For plain bars under 7/16 in. in diameter a deduction from the percentages of elongation prescribed in Table II of 0.5 per cent shall be made for each decrease of 1/32 in. of the specified diameter below 7/16 in.

(*f*) For No. 3 deformed bar (nominal diameter ⅜ in.) a deduction from the percentage of elongation prescribed in Table II of 1.00 per cent shall be made.

Bending Properties

4. (*a*) The bend test specimen shall stand being bent, at room temperature, around a pin without cracking on the outside of the bent portion. The requirements for degree of bending and sizes of pins prescribed in Table III for plain bars or Table IV for deformed bars shall be observed.

(*b*) The bend test shall be made on specimens of sufficient length to insure free bending and with apparatus which provides:

(*1*) Continuous and uniform application of force throughout the duration of the bending operation,

(*2*) Unrestricted movement of the specimen at points of contact with the apparatus, and

RAIL-STEEL BARS FOR CONCRETE REINFORCEMENT

(3) Close wrapping of the specimen around the pin or mandrel during the bending operation.

TABLE III.—BEND TEST REQUIREMENTS FOR PLAIN BARS.[a]

Diameter of Bar, in.	Bend Test Requirement
Under ¾	180 deg. $d = 4t$
¾ and over	90 deg. $d = 4t$

[a] On plain bars whose application is in unbent form such as load transfer dowels, the cold bend test shall be waived.

NOTE.—d = diameter of pin around which the specimen is bent, and
t = diameter of the specimen.

TABLE IV.—BEND TEST REQUIREMENTS FOR DEFORMED BARS.

Bar Designation Number	Bend Test Requirement
Under No. 6 (nominal diameter ¾ in.)	90 deg. $d = 6t$
No. 6 and over	90 deg. $d = 6t$

NOTE.—d = diameter of pin around which the specimen is bent, and
t = diameter of the specimen.

(c) Other methods of bend testing may be used, but failures due to such methods shall not constitute a basis for rejection.

Test Specimens

5. Tension and bend test specimens from plain or deformed bars shall be of the full section of bars as rolled. For tension tests of deformed bars the sectional area used for unit stress determination shall be calculated from the length and weight of the test specimen (Note).

NOTE.—The area in square inches may be calculated by dividing the weight per linear inch of specimen in pounds by 0.2833 (weight of 1 cu. in. of steel), or by dividing the weight per linear foot of specimen in pounds by 3.4 (weight of steel 1 in. square and 1 ft. long).

Number of Tests

6. (a) One tension test and one bend test shall be made from each lot of ten tons or fraction thereof of each size or bar designation number rolled from rails varying not more than 10 lb. per yd. in nominal weight.

(b) If any test specimen develops flaws, it may be discarded and another specimen substituted.

RAIL-STEEL BARS FOR CONCRETE REINFORCEMENT

TABLE V.—PERMISSIBLE VARIATIONS FROM THEORETICAL WEIGHTS.

NOTE.—The theoretical weights for deformed bars listed in Table I and the established standard weights for plain bars rolled to fractions of inches shall be used to establish conformance to this table.

Diameters of Bars, in.	Permissible Variations from Theoretical Weights	
	Lot,[a] Over or Under, per cent	Individual Bar, Under, per cent
Plain bars:		
Under 3/8	5	10
3/8 and over	3.5	6
Deformed bars, all sizes	3.5	6

[a] The term "lot" means all the bars of the same nominal weight per linear foot in a carload.

(*c*) If the percentage of elongation of any tension test specimen is less than that specified in Section 3 and any part of the fracture is outside the middle third of the gage length, as indicated by scribe scratches marked on the specimen before testing, a retest shall be allowed.

Permissible Variations in Weight

7. The permissible variations in weight shall not exceed the limits prescribed in Table V.

Finish

8. The bars shall be free from injurious defects and shall have a workmanlike finish.

Marking

9. The brand of the manufacturer shall be legibly rolled on all deformed bars. For the purpose of identification, a distinctive pattern is considered to be a manufacturer's brand. When loaded for mill shipment, all bars shall be properly separated and tagged with the manufacturer's test identification number.

Inspection

10. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works that concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in

RAIL-STEEL BARS FOR CONCRETE REINFORCEMENT

accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection

11. Material that shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Those interested in Federal Work should obtain Federal Specification QQ-B-71

Tentative Specifications for

MINIMUM REQUIREMENTS FOR THE DEFORMATIONS OF DEFORMED STEEL BARS FOR CONCRETE REINFORCEMENT [1]

ASTM Designation: A 305–56 T

ISSUED, 1950; Revised, 1953, 1956.[2]

These Tentative Specifications have been approved by the sponsoring committee and accepted by the Society in accordance with established procedures, for use pending adoption as standard. Suggestions for revisions should be addressed to the Society at 1916 Race St., Philadelphia 3, Pa.

Scope

1. These requirements are intended to define the surface deformations on deformed concrete reinforcement bars. Nothing herein is intended to conflict in any way with the Specifications for Steel Bars for Concrete Reinforcement (ASTM Designations: A 15, A 16, and A 160).[3]

Definitions

2. (*a*) As used within the scope and intent of these requirements, the term "Deformed Concrete Reinforcing Bar" shall mean any deformed steel bar intended for use as reinforcement in reinforced concrete construction, which conforms to the Specifications for Billet-Steel Bars for Concrete Reinforcement (ASTM Designation: A 15),[3] Rail-Steel Bars for Concrete Reinforcement (ASTM Designation: A 16),[4] or Axle-Steel Bars for Concrete Reinforcement (ASTM Designation: A 160), and of which the surface is provided with lugs or protrusions (hereinafter called "deformations") which (*1*) inhibit longitudinal movement of the bar relative to the concrete which surrounds the bar in such construction and (*2*) conform to the provisions of Section 3.

(*b*) The term "bar number" as used herein refers to the numerical designations of the bars as tabulated in Table I under the column headed "Bar Designation Number."

Requirements

3. (*a*) Deformations shall be spaced along the bar at substantially uniform distances. The deformations on opposite sides of the bar shall be similar in size and shape.

(*b*) The deformations shall be placed with respect to the axis of the bar so that the included angle is not less than 45 deg. Where the line of deformations forms an included angle with the axis of the bar of from 45 to and including 70 deg., the deformations shall alternately reverse in direction on each side, or those on one side shall be reversed in direction from those on

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[1] Under the standardization procedure of the Society, these specifications are under the jurisdiction of the ASTM Committee A-1 on Steel.

[2] Latest revision accepted by the Administrative Committee on Standards, May 2, 1956.

Prior to their present publication as tentative, these specifications were published as tentative from 1947 to 1949, being adopted in 1949 and published as standard from 1949 to 1950.

[3] See page 398.

[4] See page 403.

MINIMUM REQUIREMENTS FOR THE DEFORMATIONS OF DEFORMED STEEL BARS FOR CONCRETE REINFORCEMENT

the opposite side. Where the line of deformations is over 70 deg., a reversal in direction is not required.

(*c*) The average spacing or distance between deformations on each side of the bar shall not exceed seven-tenths of the nominal diameter of the bar.

(*d*) The overall length of deformations shall be such that the gap between the extreme ends of the deformations on opposite sides of the bar shall not exceed 12½ per cent of the nominal perimeter of the bar. Where the extreme ends terminate in a longitudinal rib, the width of the longitudinal rib shall be considered the gap. Where more than two longitudinal ribs are involved, the total width of all longitudinal ribs shall not exceed 25 per cent of the nominal perimeter of the bar; furthermore, the summation of gaps shall not exceed 25 per cent of the nominal perimeter of the bar. The nominal perimeter of the bar shall be 3.14 times the nominal diameter.

TABLE I.—DIMENSIONAL REQUIREMENTS FOR DEFORMED STEEL BARS FOR CONCRETE REINFORCEMENT.

Deformed Bar Designation Number [a]	Unit Weight, lb. per ft.	Nominal Dimensions, Round Sections			Deformation Requirements		
		Diameter, in.	Cross-Sectional Area, sq. in.	Perimeter, in.	Maximum Average Spacing, in.	Minimum Height, in.	Maximum Gap (Chord of 12½ per cent of Nominal Perimeter), in.
3	0.376	0.375	0.11	1.178	0.262	0.015	0.143
4	0.668	0.500	0.20	1.571	0.350	0.020	0.191
5	1.043	0.625	0.31	1.963	0.437	0.028	0.239
6	1.502	0.750	0.44	2.356	0.525	0.038	0.286
7	2.044	0.875	0.60	2.749	0.612	0.044	0.334
8	2.670	1.000	0.79	3.142	0.700	0.050	0.383
9 [b]	3.400	1.128	1.00	3.544	0.790	0.056	0.431
10 [b]	4.303	1.270	1.27	3.990	0.889	0.064	0.487
11 [b]	5.313	1.410	1.56	4.430	0.987	0.071	0.540

[a] Bar numbers are based on the number of eighths of an inch included in the nominal diameter of the bars. The nominal diameter of a deformed bar is equivalent to the diameter of a plain bar having the same weight per foot as the deformed bar.

[b] Bars of designation Nos. 9, 10, and 11 correspond to the former 1-in. square, 1⅛-in. square, and 1¼-in. square sizes and are equivalent to those former standard bar sizes in weight and nominal cross-sectional areas.

(*e*) The average height of deformations shall be not less than the following percentages of the nominal diameter of the bar:

MINIMUM REQUIREMENTS FOR THE DEFORMATIONS OF DEFORMED STEEL BARS FOR CONCRETE REINFORCEMENT

Deformed Bar Designation Number	Minimum Height of Deformations, per cent of nominal diameter of bar
3	4
4	4
5	4½
6 to 11 incl.	5

(*f*) The spacing, height, and gap of deformations shall conform to the requirements prescribed in Table I.

Measurements

4. (*a*) The average spacing of deformations shall be determined by dividing a measured length of the bar specimen by the number of individual deformations and fractional parts of deformations on any one side of the bar specimens. A measured length of the bar specimen shall be considered the distance from a point on a deformation to a corresponding point on any other deformation on the same side of the bar.

(*b*) The average height of deformations shall be determined from measurements made on not less than two typical deformations. Determinations shall be based on three measurements per deformation, one at the center of the overall length and the other two at the quarter points of the overall length.

Number of Test Specimens

5. To indicate adequately the conformity to these requirements, one bar specimen shall be obtained from each ten tons or fraction thereof of the lot.[4]

Rejection

6. Insufficient height, insufficient circumferential coverage, or excessive spacing of deformations shall not constitute cause for rejection unless it has been clearly established by determinations on each lot [4] that typical deformation height, gap, or spacing do not conform to the minimum requirements prescribed in Section 3. No rejection may be made on the basis of measurements if fewer than ten adjacent deformations on each side of the bar are measured.

[4] A lot is defined as all the bars of one bar number and pattern of deformation.

Standard Specifications for
COLD-DRAWN STEEL WIRE FOR CONCRETE REINFORCEMENT[1]
ASTM Designation: A 82–34
Adopted, 1927; Revised, 1933, 1934[2]

Reaffirmed in 1946 Without Change.

This Standard of the American Society for Testing Materials is issued under the fixed designation A 82; the final number indicates the year of original adoption as standard or, in the case of revision, the year of last revision.

Scope

1. These specifications cover cold-drawn steel wire to be used as such, or in fabricated form, for the reinforcement of concrete, in gages not less than 0.080 in. nor greater than 0.625 in.

Basis of Purchase

2. When wire is ordered by gage number, the following relation between the gage number and the diameter in inches shall apply, unless otherwise specified:

Gage Number	Equivalent Diameter, in.	Gage Number	Equivalent Diameter, in.
0000000	0.4900	5	0.2070
000000	0.4615	6	0.1920
00000	0.4305	7	0.1770
0000	0.3938	8	0.1620
000	0.3625	9	0.1483
00	0.3310	10	0.1350
0	0.3065	11	0.1205
1	0.2830	12	0.1055
2	0.2625	13	0.0915
3	0.2437	14	0.0800
4	0.2253		

Process

3. (*a*) The steel shall be made by one or more of the following processes: open-hearth, electric-furnace, or acid-bessemer.

(*b*) The wire shall be cold drawn from rods that have been hot rolled from billets.

Tensile Properties

4. (*a*) The material, except as specified in Paragraphs (*b*) and (*c*), shall conform to the following requirements as to tensile properties:

Tensile strength, min., psi	80 000
Yield point, min., psi	0.8 tens. str.
Reduction of area, min., per cent	30

(*b*) For material to be used in the fabrication of mesh, a minimum tensile strength of 70,000 psi. shall be permitted.

[1] Under the standardization procedure of the Society, these specifications are under the jurisdiction of the A.S.T.M. Committee A-1 on Steel.

[2] Prior to adoption as standard, these specifications were published as tentative from 1921 to 1927, being revised in 1927.

COLD-DRAWN STEEL WIRE FOR CONCRETE REINFORCEMENT

(*c*) For material testing over 100,000 psi. tensile strength, the reduction of area shall be not less than 25 per cent.

(*d*) The yield point shall be determined by the drop of the beam or halt in the gage of the testing machine. In case no definite drop of the beam or halt in the gage is observed until final rupture occurs, the test shall be construed as meeting the requirement for yield point in Paragraph (*a*).

Bending Properties

5. The bend test specimen shall stand being bent cold through 180 deg. without cracking on the outside of the bent portion, as follows:

Diameter of Wire	Bend Test
0.3 in. or under	bend around a pin the diameter of which is equal to the diameter of the specimen
Over 0.3 in.	bend around a pin the diameter of which is equal to twice the diameter of the specimen

Test Specimens

6. Tension and bend test specimens shall be of the full section of the wire as drawn.

Number of Tests

7. (*a*) One tension test and one bend test shall be made from each ten tons or less of each size of wire.

(*b*) If any test specimen shows defects or develops flaws, it may be discarded and another specimen substituted.

Permissible Variations in Gage

8. The dimensions of the wire, on any diameter, shall not vary more than plus or minus 0.003 in. from the specified nominal diameter. The difference between the maximum and minimum diameters, as measured on any given cross-section of the wire, shall not be more than 0.003 in.

Finish

9. The wire shall be free from injurious defects and shall have a workmanlike finish with smooth surface.

Inspection

10. The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection

11. Material which shows injurious defects subsequent to its acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Tentative Specifications for
WELDED STEEL WIRE FABRIC FOR CONCRETE REINFORCEMENT[1]
ASTM Designation: A 185–56 T

ISSUED, 1953; Revised, 1954, 1956.[2]

These Tentative Specifications have been approved by the sponsoring committee and accepted by the Society in accordance with established procedures, for use pending adoption as standard. Suggestions for revisions should be addressed to the Society at 1916 Race St., Philadelphia 3, Pa.

Scope

1. These specifications cover welded wire fabric to be used for the reinforcement of concrete.

Description

2. The term "welded wire fabric" as herein used designates a material composed of cold-drawn steel wires fabricated into sheet (or so-called "mesh") formed by the process of electric welding. The finished material shall consist essentially of a series of longitudinal and transverse wires arranged substantially at right angles to each other and welded together at all points of intersection.

Grade of Wire

3. The wire used in the manufacture of welded wire fabric shall conform to the Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement (ASTM Designation: A 82).[3]

Fabrication

4. (*a*) The wires shall be assembled by automatic machines or by other suitable mechanical means which will assure accurate spacing and alignment of all members of the finished fabric.

(*b*) Longitudinal and transverse members shall be securely connected at every intersection by a process of electrical-resistance welding which employs the principle of fusion combined with pressure.

(*c*) Wire of proper grade and quality when fabricated in the manner herein required shall result in a strong, serviceable mesh-type product having substantially square or rectangular openings. It shall be fabricated and finished in a workmanlike manner, shall be free from injurious defects, and shall conform to these specifications.

Mechanical Properties

5. (*a*) All wire of the finished fabric shall meet the minimum requirements for tensile properties and shall also withstand the bend test as prescribed for the wire before fabrication in the Standard Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement (ASTM Designation: A 82).[3]

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[1] Under the standardization procedure of the Society, these specifications are under the jurisdiction of the ASTM Committee A-1 on Steel.

[2] Latest revision accepted by the Administrative Committee on Standards May 2, 1956.

Prior to their present publication as tentative, these specifications were published as tentative from 1936 to 1937. They were published as standard from 1937 to 1956. Tentative Specifications were issued as a revision of the Standard, and published from 1953 to 1956, being revised in 1954 and 1956.

[3] See page 411.

WELDED STEEL WIRE FABRIC FOR CONCRETE REINFORCEMENT

(*b*) In order to assure adequate weld shear strength between longitudinal and transverse wires, weld shear tests as described in Section 6 (*c*) shall be made. The minimum average shear value of the weld in pounds for fabric having a wire size differential of up to and including four gages shall be not less than 35,000 multiplied by the area of the longitudinal wire in square inches. Typical examples of a wire size differential of four gages are as follows:

Longitudinal	Transverse
No. 0 gage	No. 4 gage
No. 2 gage	No. 6 gage

(*c*) Fabric having a wire gage differential between longitudinal and transverse wires of five or greater shall not be subject to a weld shear requirement.

Tension Tests and Weld Shear Tests

6. (*a*) Tests for determination of conformance to the requirements of Section 5 (*a*) may be made on the mesh after fabrication either across or between the welds.

(*b*) Reduction of area may be determined by measuring the ruptured section of a specimen which has been tested either across or between the welds. However, in the case of a specimen which has been tested across a weld, the measurement shall be made only when rupture has occurred at a sufficient distance from the center of the weld to permit an accurate measurement of the fractured section.

(*c*) Weld shear tests for determination of conformance to the requirements of Section 5 (*b*) shall be conducted using a fixture of such design as to prevent rotation of the transverse wire. The transverse wire shall be placed in the anvil of the testing device which is secured in the tensile machine and the load then applied to the longitudinal wire. Four welds selected at random from a specimen representing the entire weld of the fabric, exclusive of the selvage wire, shall be tested for weld shear strength.

The lot shall be deemed to conform to the requirements for weld shear strength if the average of the four samples is equal to, or exceeds 35,000 psi. If this average fails to meet the prescribed minimum value, all the welds across the specimen shall then be tested. The fabric will be acceptable if the average of all weld shear test values across the specimen meets the prescribed minimum value.

Bend Tests

7. The bend test shall be made on a specimen between the welds.

Test Specimens

8. (*a*) Test specimens for testing tensile properties shall be obtained by cutting from the finished fabric, units of suitable size to enable proper performance of the intended tests.

(*b*) Specimens used for testing tensile properties across a weld shall have the welded joint located approximately at the center of the strand being tested, and the cross wire forming the welded joint shall extend approximately 1 in. beyond each side of the welded joint.

(*c*) Test specimens for determining weld shear properties shall be obtained by cutting from the finished fabric a section, including one transverse wire, across the entire width of the sheet or roll. From this specimen, four welds exclusive of the selvage, shall be selected at random for testing.

(*d*) Tests for conformance to dimensional characteristics shall be made on full sheets or rolls.

(*e*) If any test specimen shows defects or develops flaws it may be discarded and another substituted.

Number of Tests

9. (*a*) One test for conformance with the provisions of Section 5 (*a*) shall be made for each 75,000 sq ft of fabric or fraction thereof.

(*b*) One specimen for each 300,000 sq ft of fabric or fraction thereof as defined in Section 8 (*c*) shall be tested for conformance to the requirements of Section 5 (*b*).

Gages, Spacing, and Dimensions

10. Gages, spacing, and arrangement of wires, and dimensions of units in flat sheet form or rolls shall conform to the requirements specified by the purchaser.

Width of Fabric

11. (*a*) The width of fabric shall be considered to be the center-to-center distance between outside longitudinal wires, unless otherwise specified.

(*b*) Transverse wires shall not project beyond the centerline of each longitudinal edge wire more than a distance of 1 in., unless otherwise specified.

Permissible Variations in Wire Diameter

12. The permissible variation in diameter of any wire in the finished fabric shall conform to the tolerances prescribed for the wire before fabrication in the ASTM Specifications A 82.[1]

Spacings

13. The average spacing of wires shall be such that the total number contained in a sheet or roll is at least equal to that determined by the specific spacing, but the center-to-center distance between individual wires may vary not more than ¼ in. from the specified spacing.

Over all Dimensions

14. (*a*) The over all length of flat sheets, measured on any wire, may vary ± 1 in. or 1 per cent, whichever is greater.

(*b*) In case the width of flat sheets or rolls is specified as the over all width (tip-to-tip length of cross wires), the width shall not be more than 1 in. greater or less than the specified width.

Rolls or Sheets

15. Welded wire fabric may be furnished either in flat sheets or in rolls as specified by the purchaser. Rolls of fabric made of 10 gage wire or larger shall be furnished with a core diameter of not less than 10 in.

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[1] See page 411.

WELDED STEEL WIRE FABRIC FOR CONCRETE REINFORCEMENT

Bundling

16. (*a*) When fabric is furnished in flat sheets, it shall be assembled in bundles of convenient size containing not more than 150 sheets aud securely fastened together.

(*b*) When fabric is furnished in rolls, each roll shall be secured so as to prevent unwinding during shipping and handling.

Marking

17. Each bundle of flat sheets and each roll shall have attached thereto a suitable tag bearing the name of the manufacturer, description of the material and such other information as may be specified by the purchaser.

Inspection

18. The inspector representing the purchaser shall have free entry at all times while work on the contract of the purchaser is being performed to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, without charge, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests and inspection shall be made at the place of manufacture prior to shipment, unless otherwise specified, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

Rejection and Retests

19. (*a*) Material which does not meet the requirements of these specifications may be rejected. Unless otherwise specified, any rejection shall be reported to the manufacturer within five days from the time of selection of test specimens.

(*b*) In case a specimen fails to meet the tension, or bend test, the material shall not be rejected until two additional specimens taken from other wires in the same sheet or roll, have been tested. The material shall be considered as meeting these specifications in respect to any prescribed tensile property, provided the tested average for the three specimens, including the specimen originally tested, is at least equal to the required minimum for the particular property in question and provided further that none of the three specimens develops less than 80 per cent of the required minimum for the tensile property in question. The material shall be considered as meeting these specifications in respect to bend test requirements, provided both additional specimens satisfactorily pass the prescribed bend test.

(*c*) Any material which shows injurious defects subsequent to its acceptance at the manufacturer's works may be rejected and the manufacturer shall be promptly notified.

(*d*) Welded joints shall withstand normal shipping and handling without becoming broken; but the presence of broken welds, regardless of cause, shall not constitute cause for rejection unless the number of broken welds per sheet exceeds 4 per cent of the total number of joints in a sheet, or if the material is furnished in rolls, 4 per cent of the total number of joints in 150 sq ft of fabric and, furthermore, provided not more than one-half the permissible maximum number of broken welds are located on any one wire.

(*e*) In case rejection is justified, by reason of failure to meet the weld

shear requirements, four additional specimens shall be taken from four different sheets or rolls and tested in accordance with Section 6 (*c*). If the average of all the weld shear tests does not meet the requirement the lot shall be rejected.

In case rejection is justified by reason of failure to meet the requirements for dimensions, the amount of material rejected shall be limited to those individual sheets or rolls which fail to meet these specifications. If, however, the total number of sheets or rolls thus rejected exceeds 25 per cent of the total number in the shipment, the entire shipment may be rejected.

Rehearing

20. Rejected materials shall be preserved for a period of at least two weeks from the date of inspection, during which time the manufacturer may make claim for a rehearing and retesting.

SPECIFICATIONS FOR CONCRETE WORK

The following specifications are not intended to be all-inclusive or absolutely complete, but they are intended to give the specification writer who may not have material in this particular field at his fingertips a typical arrangement that is suitable for the average small reinforced concrete framed structure. The specification writer can make adjustments for weather conditions, exposures, type of construction or local materials or workmanship, but there may be some advantage in an outline of this sort for convenient reference.

C1—General. All bidders are referred to the "General Conditions of the Contract," which shall be a part of this specification.

C2—Scope. This Contractor shall furnish all materials, equipment and labor necessary to construct all plain and reinforced concrete work, including footings, foundation walls, slabs on the ground, and any structural concrete, as well as any drain racks, pits (*state any other similar items*), floor fills or any other miscellaneous concrete either of a decorative or utilitarian character.

C3—Lines and Levels. This Contractor shall employ an experienced surveyor to pick up the reference points established under "Excavation" and to supply all necessary lines and levels to insure that all finished concrete work is straight, true and square.

C4—Portland Cement. All cement used for structural and architectural concrete work shall be portland cement conforming to the American Society for Testing Materials Spec. C150 (*latest*), types I or III, or air-entraining portland cement, ASTM C175 (*latest*), type IA, or IIIA.

Cement shall be delivered on the job in bags containing one cubic foot (approx. 94 lbs.) each (*unless a special arrangement to use bulk cement has been developed*). Each consignment of cement shall be so piled as to be segregated from every other consignment and shall be housed in a waterproof shed and stored on a floor or platform above the general ground level, and shall be well protected from dampness. No cement which has partially hardened or been otherwise damaged shall be used on this job. Retempering of cement shall not be permitted under any circumstances.

C5—Fine Aggregate. Fine aggregate shall preferably be sand, and particles shall be coarse, sharp and clean. Limestone screenings, pulverized rock, or fine gravel will not be accepted. Sand shall be free from dust, loam, dirt, vegetable matter, or any other foreign or deleterious material. When dry, sand shall all pass a screen having ¼″ square meshes and not more shall pass a 100-mesh screen than a maximum of 6 per cent. Decantation tests may be made to limit the amount of loam.

Aggregates, in addition to these specifications, shall meet the requirements of ASTM C33 (*latest*). (*If the use of light weight aggregates is contemplated, refer to ASTM C330* (*latest*).)

C6—Coarse Aggregate. Aggregate used in concrete work shall be either screened, crushed rock, or natural gravel, washed and graded, or blast-furnace slag. In any case, coarse aggregate shall be regularly graded from a maximum

of 1″ (*or state maximum size acceptable*) down to a minimum of ¼″, and must be clean, hard and durable, free from any long, splintery pieces (*or a maximum of five per cent by weight*), and free from dust, dirt, vegetable or organic matter. Mixed aggregates will not be permitted, such as, crusher run stone or bank-run gravel, because of the uneven ratio of fine to coarse materials. Coarse aggregates shall be cleaned, screened and regraded for uniformity.

C7—Water. It is anticipated that tap-run water will be used for mixing concrete, but any water that is potable shall be deemed suitable for this purpose.

C8—Admixtures. No admixtures of any kind shall be incorporated into this concrete without the written approval of the Architect (*Engineer*). As herein provided for, up to one per cent of calcium chloride by weight of cement may be added in freezing weather, but only with the written approval of the Architect (*Engineer*).

C9—Ready-mixed Concrete. In general, the use of ready-mixed concrete is favored and shall conform to ASTM C94. The quality of materials shall be as specified above. Care shall be taken to see that over-mixing of materials in a delayed truck does not damage the entire batch. Even with air-entraining cement, the maximum period from the time when water is added and mixing starts until the concrete is delivered into the forms shall in no case exceed 1½ hours, and the inspector shall watch the various batches so that whenever segregation or partial setting raises a question as to the quality of the concrete, it shall not be put into the forms.

C10—Reinforcing Steel. Reinforcing steel (*except for column verticals, which shall be hard grade billet, or rail steel*) shall be deformed bars meeting ASTM A15- (*latest*) for Open-Hearth, Intermediate-Grade, New-Billet Bars, or ASTM A16- (*latest*) for Rail Steel Bars. Bars shall be free from flaws, cracks or other defects of rolling, shall be true to size and shape, and shall be free of loose scales of rust. A thin coating of firmly attached rust shall not be cause for rejection. Bars shall be free from heavy dirt, paint, grease, oil, or other destroyers of bond. They shall be prefabricated to detail and delivered on the job plainly tagged and ready to set. Furnish shop detail drawings, all according to ACI 315 (*latest*) in quadruplicate and obtain approval before fabricating bars.

C11—Bar Supports. Reinforcing Steel, except for footings, shall be supported and spaced in the forms with approved types of wire bar supports meeting the requirements of the Concrete Reinforcing Steel Institute Specifications and spaced according to those recommendations.

C12—Welded Wire Fabric. Welded wire fabric shall be rectangularly welded wires of gauges and spacings specified and shall be delivered on the job in rolls and there straightened and placed. Tags designating the wire size and spacing shall be left on each roll until ready for use. Welded wire fabric shall have end laps of one full mesh tip to tip of longitudinal wires and edge lap obtained by overlapping longitudinal selvage wires and wiring all laps securely together. Tuck ends of welded wire mesh well down into edge beams or walls. Do not leave unreinforced border strips.

C13—Measuring Concrete Materials. The method of measuring the materials, including water for concrete or mortar, shall be one which will insure separate and uniform proportion of each of the materials at all times, controlling by weight.

C14—Inspection and Testing. All reinforcing steel shall have certified mill test reports as to its chemical and physical properties. (*If independent laboratory tests are desired elaborate.*)

This contractor shall include the sum of $——— to employ a competent independent laboratory satisfactory to the Architect (*Engineer*) to select fine and coarse aggregates, make trial mixes, and establish proportions for each type of concrete on this job; and to instruct the contractor upon the obtaining, handling and delivering of test cylinders; and to test and report upon the quality of the concrete actually used. All sampling and testing shall follow ASTM standards. At least six cylinders shall be taken from each full day's concreting or from each 150 cu. yd. (*or appropriate to size of job*) of concrete placed, three for a seven-day test and three for a twenty-eight-day test.

C15—Proportions. Concrete shall be proportioned by the water-cement ratio method. The proportioning of materials shall be based on the requirements for a plastic and workable mix with the use of not less than 5-½ sacks of cement per cubic yard and not more than 6-½ gallons of water per sack of cement, including the surface water carried by the aggregate. The proportion of fine to coarse aggregate shall be adjusted to produce maximum workability, but in no case shall the ratio of fine to coarse aggregate be less than ½ nor more than 1, i.e. the fine aggregate shall be ⅓ to ½ and the coarse aggregate ½ to ⅔ of the total fine and coarse aggregate. Concrete shall be placed with a slump of approximately 4″ if manually spaded into place and 3″ if internal vibrators are used.

Concrete shall develop an ultimate compressive strength, f'_c, of at least 3000 psi in standard 6″ x 12″ cylinders at 28 days, moist cured in the laboratory. (*If 2500 psi or other strength is acceptable for walls, footings, or in other places, elaborate here.*)

C16—Mixing. All concrete shall be mixed by machine. The ingredients of the concrete shall be mixed to the required consistency herein specified and the mixing shall be continued at least 1-½ minutes after all materials are in the drum before any part of the batch is discharged from the drum. The drum shall be completely emptied before receiving materials for the succeeding batch. The volume of the mixed materials for the batch should not exceed the manufacturer's rated capacity for the drum in cubic feet of mixed materials.

The mixer must be equipped with a water storage and measuring device (which can be locked) and with a suitable charging hopper. Ready-mixed concrete may be used if it complies with all the requirements of this specification.

C17—Forms. If the earth will stand reasonably vertical and firm during excavating and concreting, no forms need be constructed for isolated footings or footing courses.

Forms for walls below grade may be built of used lumber (or of prefabricated standard panels) and shall be tight enough to prevent leakage of concrete. They shall be straight enough to maintain reasonably straight surfaces.

Forms for walls that will be exposed in the finished building shall have the surface in contact with the exposed concrete formed with plywood, Presdwood, Celotex form liner or other suitable material for producing smoothly finished surfaces, or surfaces of specified texture.

Forms for concrete joist construction may consist of a series of inverted troughs of metal or wood properly supported and with suitable soffits, designed to produce straight and true joists with such tapered ends or stiffening ribs as are shown or specified, but ridges or offsets caused by the overlapping of individual pieces will be acceptable provided they do not exceed $\frac{3}{16}$ in.

All formwork shall be framed with an adequate number of sufficiently stiff studs and bracing to insure against any noticeable deflection during or after pouring. Snap Ties, except for exposed architectural walls, are recommended as a means of holding opposite wall forms in place. Wire ties may be used if they are cut back an inch from the surfaces of the walls immediately after the forms are stripped and the depression pointed with cement grout. Grout shall consist of one part cement to one and one-half parts of screened sand.

Wall forms shall remain in place until the concrete has hardened sufficiently to be self-sustaining. No outside pressure should be placed on this concrete before removal of all forms.

Before any section of concrete is to be poured, this Contractor shall check with all other trades and see that all inserts, openings, bolts, sleeves, pockets and slots that may be desired are incorporated into the section about to be poured.

C18—Removal of Forms. In general, girder, beam and joist sides only and column forms may be removed within 24-48 hours after concrete is placed, and wall forms in about the same length of time provided no backfill is placed against the walls until they are adequately supported (*until basement and first floor slabs are in place and hardened*).

Girder, beam, and joist soffit forms shall remain in place with adequate shoring underneath until (a) 48 hours after floor above has been placed, (b) until test cylinders from that section have attained 75 per cent of their specified ultimate strength, f'_c, (c) until all excess construction loads have been removed, and (d), in the case of light design loads and particularly in cold weather, until the slabs above are capable of carrying the wet concrete and construction loads without aid from the floor that is to be decentered.

Approval of the Architect (*Engineer*) shall be obtained before removing any forms, but the responsibility for the preserving of straight lines, true surfaces, and a satisfactory result, including the protection of surfaces and corners from damage or abrasion are the Contractor's responsibility.

C19—Sleeves, Anchors, and Hangers. The Contractor shall consult with all other trades and ascertain where sleeves, anchors, inserts or hangers are required and shall assist these Contractors in setting same. Wherever pipes pass through walls above the floor line, neatly box out suitable openings.

The Contractor shall obtain from the various trades the sizes and locations of all pipes or ducts passing through concrete and shall provide proper form-work to leave openings to accommodate these installations.

The Contractor shall properly secure in the concrete all inserts and supports for pipe, conduit, etc. furnished by others. He shall locate and set in place all angle iron guards, anchors, expansion bolts, and any other iron or metal provisions which may be furnished by other trades.

Provide all sleeves and pockets where ducts, sewerage, plumbing, water, electrical conduits, gas or other piping of any nature may pass through foundation walls or other concrete work.

C20—Placing Reinforcement. Reinforcement shall be placed in the exact locations shown on the plans; shall, in general, be spaced and supported in the forms with bar supports equal to those specified by the Concrete Reinforcing Steel Institute in its Code of Standard Practice or the American Concrete Institute's Detailing Manual, spaced in conformity with such codes; and shall be wired together at intersections so that it will be securely and accurately held in place during the placing of concrete.

C21—Placing Concrete. Concrete shall be placed in a manner that will permit the most thorough compacting and shall be worked into all the recesses. Concrete shall be placed in its final position as soon as possible after mixing and must be in place within 1½ hours after the water has been added to the dry materials. It should be placed in one continuous operation from construction joint to construction joint.

Joints shall be formed, not simply stopped off, and such forms shall generally be perpendicular to stress lines. Construction joints are best made at points of minimum shear, as for example, midspan of slabs, joists, and beams. If joints are made at any other point, the Architect (*Engineer*) will design a shear-key of concrete with crossed reinforcing bars to develop the shear. Column joints are usually at the underside of caps or floor systems and at tops of footings, floor systems or inverted beams.

In placing, concrete shall not be allowed to drop a distance of more than six feet through free space.

Wall and column footings shall be placed in one continuous operation, i.e., caps and bases at one time; but footings shall be placed ahead of the walls or columns that they support. Build in a beveled key into the top of all wall footings on the center of walls to be carried.

Internal vibration is desirable, providing that it is not overdone. Care should be taken to keep the vibrators off the reinforcing steel. If internal vibrators are not available, hand-spading of the concrete into all recesses will be required.

Where work is stopped so that the concrete has hardened before placing is resumed, the surface shall be left level or square but roughened and covered with wet burlap. When starting again to place, clean the surface of all foreign matter and laitance, roughen up with a suitable metal tool, drench with water and slush with a thin layer of mortar made of one part cement

and two parts sand. Furnish and set dowels in all construction joints as called for on the plans or as directed by the Architect (*Engineer*).

C22—Protection of Concrete Work. Exposed surfaces shall be protected by wet burlap or canvas covering from sun, wind and rain and this shall be frequently wetted in dry and hot weather so that the entire surface is kept wet for a period of one week; or liquid curing compound satisfactory to the Architect (*Engineer*) may be used, applied as directed. When ambient temperature falls below 40°F, heat aggregates and mixing water, clear all forms, reinforcement and subgrade of snow and ice, and cover all freshly placed concrete with tarpaulins and provide heat with oil-burning salamanders or steam lines to maintain a temperature of at least 70°F for at least four days after placing concrete, and as much longer as directed by the Architect (*Engineer*). (*In northern climates and unusual exposures, elaborate in more detail.*)

C23—Defective Work. Upon removal of the forms, all stone pockets and honeycomb shall be pointed up with cement grout, one part cement to two parts sand.

C24—Pits and Machine Foundations. Form and place any pits and machine foundations shown on the plans. No exterior forms need be used if the earth will stand reasonably vertical and firm, otherwise both sides of the walls must be formed. In all cases, the interior side and any exposed concrete work must be formed. Furnish and install all anchor bolts, manhole covers and curb angles that are required in these foundations.

C25—Shop Drawings. This Contractor shall cause to be prepared and shall submit in quadruplicate and obtain approval, before fabrication, of shop drawings that clearly show the layout of all reinforcing steel of every kind. Such steel drawings shall show the bending of all bent bars, and shall be prepared strictly in accordance with ACI 315 (*latest*).

C26—Slabs on Ground. Slabs on ground are to be (*state requirements such as full 6″ thick of 3000 psi concrete reinforced with one layer of 6 x 6 8/8 welded wire fabric*).

C27—Floor Finish. Where asphalt tile finish floors occur, the Contractor shall bring the top of the concrete slab to a true level, approximately ¼ of an inch below the level of the finished tile flooring, and shall finish the surface with a steel trowel or trowelling machine to be smooth, straight and true. From then until the asphalt tile is to be placed, the Contractor shall protect the slab against marring or scratching in any way.

Where exposed concrete floor finish is to be used, this Contractor shall float the top surface of the slab smooth and true and shall finish it with a wood float to give a reasonably smooth but slightly rough-cast floated finish.

In toilets, concrete floor finish described above shall be steel trowelled to a smooth hard surface, pitched where so called for to floor drains.

All concrete finished floors shall be treated with three coats of liquid floor hardener applied according to the manufacturer's directions.

C28—Concrete Trim. Form and place all concrete trim of every description called for on the plans, including any sills, copings, base courses, steps, curbs, poured lintels, and any other items. They shall be formed neatly and to straight lines with well-braced sides, and, after stripping, the exposed surfaces shall be carborundum-rubbed sufficiently to take off any fins or excressences and to fill in any pores, after which all grout shall be rubbed off with a burlap sack.

C29—Special Work. (*Specification writer shall include such specifications for any special work such as underpinning, patching sidewalks or pavements, repairing adjoining work, building pits, tanks, vaults, hoppers, swimming pools, etc. as the size and importance of the work requires.*)

C30—Cleaning. On completion of this Contract, clean down all exposed concrete work and remove from the premises form lumber, cement sacks, and any other debris caused by this work.

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS

For Two-way Flat Slabs see pages 189-209

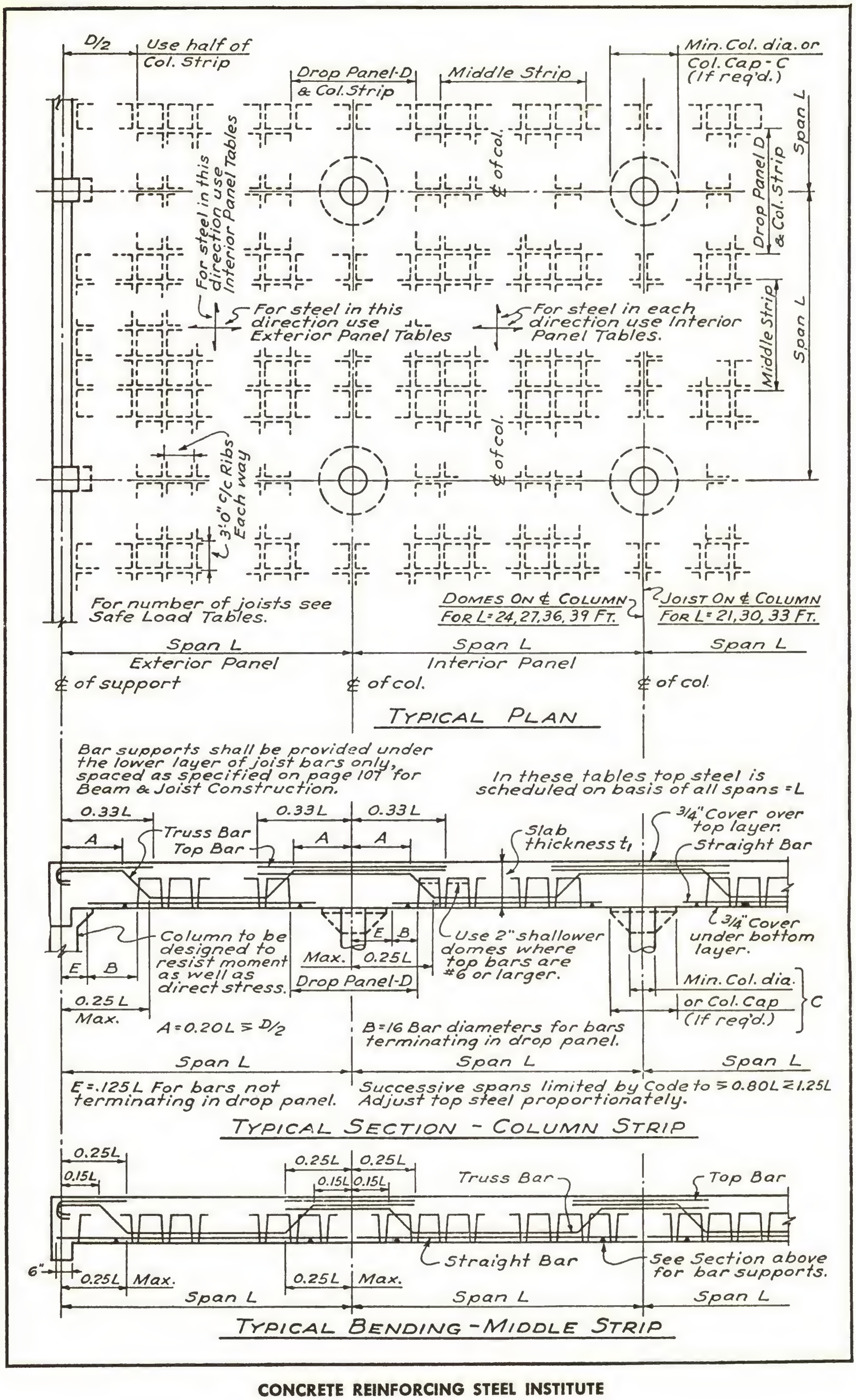
Tables are presented for height of dome, thickness of top slab, total slab thickness, size of drop panel, column or column cap, reinforcing steel in each joist, and the weight of steel and volume of concrete per square foot of floor area, for 30 in. wide domes and 6 in. wide joists on spans of 21, 24, 27, 30, 33, 36, and 39 ft, for safe superimposed loads of 50, 100, and 150 psf, for typical square interior panels, and for joists perpendicular to the wall for square exterior panels that are built integrally with exterior columns.

All tables are based upon the recommendations of the "Building Code Requirements for Reinforced Concrete (ACI 318-56)," using one set of fiber stresses, viz; $f'_c = 3000$ psi and $f_s = 20{,}000$ psi, and are based upon using deformed bars whose deformations meet the requirements of ASTM A305.

As shown on page 420, the two-way dome slab has rows of joists at right angles to each other, with domes omitted around the columns to form "drop panels," and either the columns are of sufficient size to keep the diagonal tension within allowable values or a flaring cap is provided. Shallower domes are sometimes used around the drop panel to provide space for heavy top bars. Joists in each direction are divided into two bands or strips, one over the columns and extending the width of the drop panel, designed to provide reinforcement for the column strip, and the other filling in between consecutive column strips and designed to provide reinforcement for the middle strip. A second set of similar strips of joists runs at right angles to the first. Each joist has a straight bottom bar and a truss bar which is in the bottom of the slab between column centers and bent up into the top of the slab on each column center-line. When shears exceed the relatively low values allowed for v_c, flat, welded, ladder-like stirrups are added, usually just for the length of a single dome. Additional straight bars are required both ways in the top of each drop panel. A layer of 6 x 6-#6/#6 welded wire fabric is recommended in the top slab over the domes in all cases, and this is to be increased or supplemented if unusually heavy concentrated loads are contemplated.

The structure must have at least three consecutive panels in a row in each direction to come within the ACI Code values for moments; if the building is narrower a special analysis must be made which will have the effect of increasing the reinforcement. The successive spans must be of such lengths that they do not differ by more than twenty per cent of the longer span.

The panels tabulated are in multiples of 3 ft so that joists space out exactly, spans of 21, 30, and 33 ft having a joist on the column center and spans of 24, 27, 36, and 39 ft having a row of domes on the column center, arranged this way to provide the proper size of drop panel. Narrower domes are available to fill out column spacings that are not multiples of 3 ft, and suppliers vary considerably in standards (there being no U. S. Department of Commerce Standard Practice, as yet), so the designer will have to obtain detailed information as to what is most readily available at any given job site.


D/2
Use half of Col. Strip
Drop Panel-D & Col. Strip
Middle Strip
Min. Col. dia. or Col. Cap - C (If req'd.)
Span L
Drop Panel D & Col. Strip
Middle Strip
Span L
℄ of col.
For steel in this direction use Interior Panel Tables
For steel in this direction use Exterior Panel Tables
For steel in each direction use Interior Panel Tables.
3'0" c/c Ribs Each way
℄ of col.
For number of joists see Safe Load Tables.
Domes on ℄ Column For L = 24, 27, 36, 39 Ft.
Joist on ℄ Column For L = 21, 30, 33 Ft.
Span L
Exterior Panel
Span L
Interior Panel
Span L
℄ of support
℄ of col.
℄ of col.
Typical Plan
Bar supports shall be provided under the lower layer of joist bars only, spaced as specified on page 107 for Beam & Joist Construction.
In these tables top steel is scheduled on basis of all spans = L
0.33L
0.33L
0.33L
A
Truss Bar
Top Bar
A
A
Slab thickness t1
3/4" Cover over top layer.
Straight Bar
3/4" Cover under bottom layer.
Column to be designed to resist moment as well as direct stress.
E
B
Max.
0.25L
Drop Panel-D
Use 2" shallower domes where top bars are #6 or larger.
E
B
0.25 L
Max.
Min. Col. dia. or Col. Cap (If req'd.)
C
A = 0.20L ≥ D/2
B = 16 Bar diameters for bars terminating in drop panel.
Span L
Span L
Span L
E = .125 L For bars not terminating in drop panel.
Successive spans limited by Code to ≥ 0.80L ≤ 1.25L
Adjust top steel proportionately.
Typical Section - Column Strip
0.25L
0.15L
0.25L
0.25L
0.15L
0.15L
Truss Bar
Top Bar
Straight Bar
See Section above for bar supports.
6"
0.25L
Max.
0.25L
Max.
Span L
Span L
Span L
Typical Bending - Middle Strip

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS

While values have been given only for square panels, it is possible to estimate values for a rectangular panel by using the long side for one set of joists and the short side for the other. The ACI Code limits the ratio of long-to-short side to 1.33. Because of the considerable width of domes and the two-way nature of the slab, the designer would do well to sketch out the spacings for a typical panel and correlate with the column spacings as a part of the early planning.

For exterior panels, it is possible to take the joists that are continuous, i.e., parallel to the discontinuous edge, from the tables for Typical Interior Panels. The strips that are noncontinuous, i.e., perpendicular to the discontinuous edge, are given by the table for Strips Perpendicular to an Exterior Wall, provided that the exterior edge of the panel frames into a concrete column or concrete bearing wall integral with the slab. If the slab simply rests upon a masonry wall without any edge restraint, then the bars in the joists perpendicular to the exterior wall must be changed from the values in the table for Typical Exterior Panels as follows:—

Positive steel in column strip:—Increase 50%.
Negative steel in column strip at wall:—Decrease to 17% of tabulated value.
Negative steel in column strip over first interior column:—Increase 30%.
Positive steel in middle strip:—Increase 30%.
Negative steel in middle strip at exterior wall:—Decrease to 60% of tabulated value.
Negative steel in middle strip at first interior row of columns:—Increase 33%.

For corner panels which are discontinuous on two edges, both sets of strips should be taken from the tables for Typical Exterior Panels, and if both discontinuous edges rest upon masonry walls, the corrections above shall apply to each set of strips.

To guard against excessive diagonal tension stresses in the slab around the column head, the tables give the minimum diameter of support (i.e., column or cap) that will keep $v_c \lessapprox 0.03f'_c$ (ACI 1002(c) 2) when 50 per cent of the total negative flexural reinforcement is within the periphery, or $v_c \lessapprox 0.025f'_c \lessapprox 85$ psi when 25 per cent is so located, and prorated for intermediate percentages. For moderate loads and spans, such sizes are not excessive as column diameters, but for heavier loads on longer spans a conical cap seems desirable. These tables do not consider the use of any of the types of "shear head" reinforcement suggested to eliminate caps.

The concrete quantities given per square foot of floor area include all structural concrete in slab and drop panel but do not include any material in the supporting beams, column capital, column, nor any floor finish above the structural slab. "Safe superimposed load" represents live load, floor finishes, partition allowance, and everything except the weight of the structural concrete. For a table of quantities in columns and column capitals, see page 106.

The weight of main steel is the average weight in pounds per square foot of all longitudinal straight and truss bars in the slab but not including stirrups, welded wire fabric, nor bars in beams, columns, walls, or footings, while the weight of stirrups is the average weight in pounds per square foot of entire slab of the stirrups alone, when these are called for in the joists.

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS

The effective depth of slab is computed on the basis of an allowance of ¾ inch cover over the bars in all cases, and, where bars cross each other, by an allowance of one bar diameter plus 0.03 in. for deformations, but the bars each way were finally selected to satisfy the greater of the steel requirements in the two directions.

While the scope of these tables is adequate for most purposes, it is not practicable to cover all possible combinations, and those which are covered must be regarded as suitable for estimating or checking purposes but requiring careful analysis for actual design problems. For those who wish to extend beyond the coverage of these tables as well as for those who wish to know how they were computed, the following example will be instructive:—

Example—For the table on page 424, design a two-way dome slab for a 30 ft square interior panel with 100 psf superimposed load, using a 14 + 3 slab (t_1 = 17 in.); stresses, $f'_c = 1350$ psi, $f_s = 20{,}000$ psi, $n = 10$.

Lay out the domes as 10 rows of 10 domes less 4 x 4 for drop panel or 84 total, each of which displaces 970 lb of concrete (table on page 423).

$$30 \times 30 \;@ \begin{cases} 100 & \text{LL} \\ \underline{212.5} & \text{for 17'' slab} \\ 312.5 & \text{Total} \qquad = 281{,}500 \text{ lb} \end{cases}$$

$$\text{Less 84 domes @ 970 lb} = \underline{81{,}500}$$

$$W = 200{,}000 \text{ lb}$$

Shear at Edge of Drop Panel: There are 4 sides of 5 joists each or 20 joists to resist this shear.

Twenty joists @ $6 \times 15.5 \times \frac{7}{8} \times 90 = 146{,}500$ lb. The net shear is 200,000 − 12.5×12.5 @ 312.5 = 151,200 lb > 146,500, so stirrups are to be furnished along the first row of domes from the drop.

Shear at First Row Outside of Drop:—Repeating the computation to see if the stirrups need be longer than the width of a single dome, $v_c = \dfrac{200{,}000 - 18.5 \times 18.5 \times 312.5}{28 \times 6 \times \frac{7}{8} \times 15.5} = 41$ psi < 90, so stirrups are not needed past the first row of domes.

Shear around Column Head:—A sketch would suggest that one pair of joist bars and $2 \times 5 = 10$ top bars are within the periphery $C + 2(t_1 - 1\frac{1}{2}) = 26 + 2 \times 15.5 = 57$ in. This is $^{12}\!/_{28}$ = 43 per cent of all the top bars. In these tables, it is always possible to have 25% of the steel over the column permitting at least $v_c = 0.025\,f'_c = 75$ psi; it is rarely possible to have 50%, which would permit $v_c = 0.03f'_c = 90$ psi. Prorate here to v_c = 85 psi. While a direct solution might be made (page 368), assume C = 26 in., then the diameter of the periphery is 57 in. = 4.75 ft. Load outside the periphery is 200,000 − $\dfrac{\pi \times 4.75^2 \times 312.5}{4} = 194{,}500$ lb, and $v_c = \dfrac{194{,}500}{\pi \times 57 \times \frac{7}{8} \times 15.5} = 80$ psi < 85 psi.

Because of the desirability of guarding against any possibility of diagonal tension failure around the column head, it is considered better to keep v_c relatively low (close to 75 psi), so C = 28 in. is investigated:—

$$v_c = \frac{200{,}000 - 0.7854 \times 4.92^2 \times 312.5}{\pi \times 59 \times \frac{7}{8} \times 15.5} = 77.5 \text{ psi, and this size will be used.}$$

Bending Moment. Because the relative stiffnesses of joists and drop panel are about the same as in a standard two-way flat slab, coefficients will be taken from ACI 1004(f).

$M_o = 0.09\,\text{WFL}\left(1 - \dfrac{2c}{3L}\right)^2$, where $F = 1.15 - \dfrac{2.33}{30} = 1.072$, so $M_o = 0.09 \times 200{,}000$ $\times 1.072 \times 30 \times 12\left(1 - \dfrac{2 \times 2.33}{3 \times 30}\right)^2 = 6{,}250{,}000$ lb-in., which is subdivided:—

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS

	Column Strip Negative	Column Strip Positive	Middle Strip Negative	Middle Strip Positive
Percentage of M_o	50	20	15	15
Moment kip—in.	3125	1250	938	938
d (in.)	15.1	15.88	15.1	15.1
A_s (sq. in.)	11.9	4.53	3.55	3.55

Positive Moment, Column Strip:—5 Joists @ 1-#6 straight and 1-#6 truss bar = 4.40 sq in. < 4.53.

Positive Moment, Middle Strip:—5 Joists @ 1-#5 straight and 1-#6 truss bar = 3.75 sq in. > 3.55.

For the entire width of panel, there is furnished 8.15 sq in. > 8.08 sq in. required.

Negative Moment, Column Strip:—

Truss bars = 2 × 5 @ 0.44 = 4.40 sq in.
Top bars = 18 @ 0.44 = 7.92

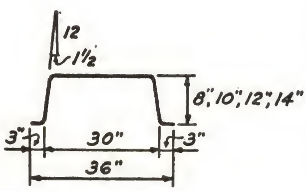
12.32 sq in. > 11.9 sq in.

Negative Moment, Middle Strip:—5 Joists @ 2 × 0.44 = 4.40 sq in. > 3.55 sq in. required. With $p = \frac{0.88}{6 \times 15.1} = 0.0097 < 0.0136$ it is unnecessary to check f_c.

Check Compression in Concrete of Drop Panel:—Use three-quarters of drop width = 92.5 in. (ACI 1002(c)1)

$$R = \frac{M}{bd^2} = \frac{3{,}125{,}000}{92.5 \times 15.1 \times 15.1} = 148 < 236, \text{ so } f_c < 1350 \text{ psi (p. 34).}$$

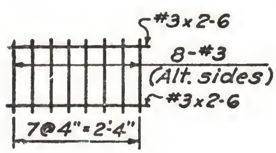
Placing of bars in two-way dome floor slabs can follow the order given on page 193.



TYPICAL DOME

DOME DATA				
Depth (in.)	Volume (cf per dome)	Weight of displaced concrete (lb per dome)	With 3″ top slab: Equiv. slab thickness (in.)	With 3″ top slab: Weight (psf)
8	3.9	580	5.8	73
10	4.8	720	6.6	83
12	5.7	850	7.4	93
14	6.5	970	8.3	104

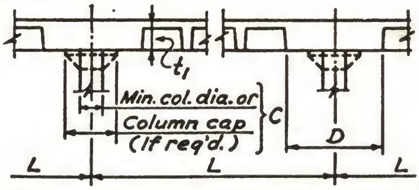
TYPICAL WELDED JOIST STIRRUP



#3 stirrups are ordinarily used.
#4 stirrups (same detail) where called for in table on pages 424-425.

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS

For two-way flat slabs see pages 189 to 209.



For general instructions and notes on the use of this table, see pages 419-423.

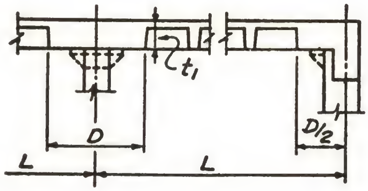
Span / Drop	Live Load (psf)	Form Depth Plus Top Slab	t_1 (in.)	Min. Col. or Cap. "C" (in.)	Interior Square Panel — Each Column Strip — No. Jst.	Each Column Strip — Bars — Trussed Straight	Each Column Strip — Bars — **Top	Each Column Strip — Stirrups per Joist*	Each Middle Strip — No. Jst.	Each Middle Strip — Bars — Trussed Straight	Wt.† of Steel (psf)
L = 21′-0″ D = 6′-6″	50	8 + 3	11	20	3	#5 x 35′-6 #4 x 16′-3	2 #4 x 14′-0 8 #4 x 12′-6	—	4	#4 x 32′-3 #4 x 21′-0	1.67
	100	8 + 3	11	20	3	#6 x 35′-6 #5 x 16′-3	2 #5 x 14′-0 8 #5 x 12′-6	2-#3	4	#4 x 32′-3 #4 x 21′-0	2.20
	150	10 + 3	13	20	3	#6 x 35′-6 #5 x 16′-3	2 #5 x 14′-0 10 #5 x 12′-6	2-#3	4	#5 x 32′-3 #4 x 21′-0	2.54
L = 24′-0″ D = 9′-6″	50	8 + 3	11	20	4	#5 x 40′-6 #5 x 16′-3	3 #5 x 16′-0 9 #5 x 14′-6	—	4	#5 x 36′-9 #4 x 24′-0	2.22
	100	10 + 3	13	20	4	#6 x 40′-6 #5 x 16′-3	3 #5 x 16′-0 9 #5 x 14′-6	2-#3	4	#5 x 36′-9 #5 x 24′-0	2.60
	150	12 + 3	15	22	4	#6 x 40′-6 #5 x 16′-3	3 #5 x 16′-0 12 #5 x 14′-6	2-#3	4	#5 x 37′-0 #5 x 24′-0	2.76
L = 27′-0″ D = 9′-6″	50	10 + 3	13	20	4	#6 x 45′-6 #5 x 19′-3	3 #5 x 18′-0 11 #5 x 16′-3	—	5	#5 x 41′-3 #4 x 27′-0	2.47
	100	12 + 3	15	22	4	#6 x 46′-0 #6 x 19′-6	3 #6 x 18′-0 11 #6 x 16′-3	2-#3	5	#5 x 41′-6 #5 x 27′-0	3.01
	150	14 + 3	17	24	4	#7 x 46′-0 #6 x 19′-6	3 #6 x 18′-0 10 #6 x 16′-3	2-#3	5	#5 x 41′-6 #5 x 27′-0	3.22
L = 30′-0″ D = 12′-6″	50	12 + 3	15	22	5	#6 x 51′-0 #5 x 19′-3	4 #6 x 20′-0 10 #6 x 18′-0	—	5	#5 x 46′-0 #5 x 30′-0	2.82
	100	14 + 3	17	28	5	#6 x 51′-0 #6 x 19′-6	4 #6 x 20′-0 14 #6 x 18′-0	2-#3	5	#6 x 46′-0 #5 x 30′-0	3.40
	150	14 + 3	17	38	5	#7 x 51′-0 #6 x 19′-6	4 #6 x 20′-0 14 #6 x 18′-0	2-#3	5	#6 x 46′-0 #6 x 30′-0	3.85
L = 33′-0″ D = 12′-6″	50	14 + 3	17	24	5	#7 x 56′-0 #6 x 22′-6	4 #6 x 22′-0 12 #6 x 20′-0	—	6	#5 x 50′-6 #5 x 33′-0	3.22
	100	14 + 3	17	38	5	#7 x 56′-0 #7 x 23′-0	4 #7 x 22′-0 12 #7 x 20′-0	2-#3	6	#6 x 50′-6 #5 x 33′-0	3.92
	150	14 + 3	17	51	5	#8 x 56′-0 #7 x 23′-0	4 #7 x 22′-0 12 #7 x 20′-0	2-#3	6	#6 x 50′-6 #6 x 33′-0	4.41
L = 36′-0″ D = 15′-6″	50	14 + 3	17	33	6	#7 x 61′-0 #6 x 22′-6	5 #6 x 24′-0 16 #6 x 21′-6	—	6	#6 x 55′-0 #6 x 36′-0	3.80
	100	14 + 3	17	50	6	#7 x 61′-0 #7 x 23′-0	5 #7 x 24′-0 15 #7 x 21′-6	2-#3	6	#7 x 55′-0 #6 x 36′-0	4.52
	150	14 + 3	17	67	6	#8 x 61′-0 #7 x 23′-0	5 #7 x 24′-0 15 #7 x 21′-6	2-#3	6	#7 x 55′-0 #7 x 36′-0	5.06
L = 39′-0″ D = 15′-6″	50	14 + 3	17	45	6	#7 x 66′-0§ #7 x 26′-0	5 #7 x 26′-0 16 #7 x 23′-6	2-#3	7	#6 x 59′-6 #6 x 39′-0	4.20
	100	14 + 3	17	63	6	#8 x 66′-0§ #7 x 26′-0	5 #7 x 26′-0 18 #7 x 23′-6	4-#3	7	#7 x 59′-6 #6 x 39′-0	4.95
	150	14 + 3	17	84	6	#8 x 66′-0§ #8 x 26′-3	6 #7 x 26′-0 23 #7 x 23′-6	4-#4	7	#7 x 59′-6 #7 x 39′-0	5.66

* 2-#3 means 1-#3 ladder stirrup (page 423) one dome length out from drop panel, each end each joist; 4-#3 means 2-#3 ladder stirrups for two dome lengths out from drop panel, each end each joist; 4-#4 means 2-#4 ladder stirrups for two dome lengths out from drop panel, each end each joist.

** Top bars are spaced evenly between truss bars of joists in column strip.

† Weight of steel per sq ft is the average weight in pounds per sq ft of all longitudinal straight, truss, and top bars in the slab but not including stirrups or welded wire fabric.

TWO-WAY DOME FLOOR SLABS—SQUARE PANELS



$f_s =$ 20,000 psi
$f_c =$ 1350 psi
$v_c =$ 75 to 90 psi
$u =$ 300 psi

Wt. of Stirrups (psf)	Av.‡ Cu. Ft. Conc. (psf)	Wt.† of Steel (psf)	Wt. of Stirrups (psf)	Strips Perpendicular to an Exterior Wall — Column Strip: Top Bars at Ext. Col.**	Column Strip: No. Jst.	Column Strip: Bars — Trussed Straight	Column Strip: Bars — Stirrups per Joist*	Column Strip: **Top bars at First Int. Col.	Middle Strip: No. Jst.	Middle Strip: Bars — Trussed Straight
—	.529	1.93	—	6 #4 x 8′-3 8 #4 x 7′-6	3	#5 x 29′-9 #5 x 16′-3	—	4 #4 x 14′-0 12 #4 x 12′-6	4	#4 x 28′-0 #4 x 21′-0
0.12	.529	2.62	0.12	6 #5 x 8′-3 6 #5 x 7′-6	3	#6 x 29′-9 #6 x 16′-3	2-#3	4 #5 x 14′-0 8 #5 x 12′-6	4	#5 x 28′-0 #5 x 21′-0
0.13	.589	2.89	0.13	6 #5 x 8′-3 8 #5 x 7′-6	3	#6 x 30′-0 #6 x 16′-3	2-#3	4 #5 x 14′-0 10 #5 x 12′-6	4	#5 x 28′-3 #5 x 21′-0
—	.544	2.48	—	6 #5 x 9′-6 6 #5 x 8′-9	4	#6 x 34′-0 #5 x 16′-3	—	3 #5 x 16′-0 9 #5 x 14′-6	4	#5 x 32′-0 #5 x 24′-0
0.14	.625	2.98	0.14	9 #5 x 9′-6 6 #5 x 8′-9	4	#6 x 34′-0 #6 x 16′-6	2-#3	3 #5 x 16′-0 12 #5 x 14′-6	4	#6 x 32′-0 #5 x 24′-0
0.15	.705	3.34	0.15	9 #5 x 9′-6 7 #5 x 8′-9	4	#7 x 34′-3 #6 x 16′-6	2-#3	3 #5 x 16′-0 14 #5 x 14′-6	4	#6 x 32′-3 #6 x 24′-0
—	.603	2.81	—	7 #5 x 10′-6 9 #5 x 9′-6	4	#6 x 38′-3 #6 x 19′-6	—	6 #5 x 18′-0 12 #5 x 16′-3	5	#5 x 36′-0 #5 x 27′-0
0.12	.694	3.44	0.12	6 #6 x 10′-6 7 #6 x 9′-6	4	#7 x 38′-6 #6 x 19′-6	2-#3	3 #6 x 18′-0 12 #6 x 16′-3	5	#6 x 36′-3 #5 x 27′-0
0.13	.776	3.80	0.13	7 #6 x 10′-6 9 #6 x 9′-6	4	#7 x 38′-9 #7 x 20′-0	2-#3	4 #6 x 18′-0 12 #6 x 16′-3	5	#6 x 36′-6 #6 x 27′-0
—	.725	3.33	—	8 #6 x 11′-9 6 #6 x 10′-9	5	#7 x 42′-6 #6 x 19′-6	—	4 #6 x 20′-0 10 #6 x 18′-0	5	#6 x 40′-0 #6 x 30′-0
0.13	.815	4.01	0.13	8 #6 x 11′-9 10 #6 x 10′-9	5	#7 x 42′-9 #7 x 20′-0	2-#3	4 #6 x 20′-0 14 #6 x 18′-0	5	#7 x 40′-3 #6 x 30′-0
0.13	.815	4.53	0.13	10 #6 x 11′-9 8 #6 x 10′-9	5	#8 x 42′-9 #7 x 20′-0	2-#3	4 #6 x 20′-0 16 #6 x 18′-0	5	#7 x 40′-3 #7 x 30′-0
—	.797	3.71	—	8 #6 x 12′-9 10 #6 x 11′-9	5	#7 x 46′-6 #7 x 23′-0	—	4 #6 x 22′-0 14 #6 x 20′-0	6	#6 x 44′-0 #6 x 33′-0
0.11	.797	4.52	0.11	8 #7 x 12′-9 8 #7 x 11′-9	5	#8 x 46′-9 #7 x 23′-0	2-#3	4 #7 x 22′-0 14 #7 x 20′-0	6	#7 x 44′-0 #6 x 33′-0
0.11	.797	5.15	0.11	10 #7 x 12′-9 10 #7 x 11′-9	5	#8 x 46′-9 #8 x 23′-3	2-#3	4 #7 x 22′-0 16 #7 x 20′-0	6	#7 x 44′-0 #7 x 33′-0
—	.820	4.37	—	8 #7 x 13′-9 10 #7 x 12′-9	6	#7 x 50′-6 #7 x 23′-0	—	5 #7 x 24′-0 15 #7 x 21′-6	6	#7 x 47′-9 #6 x 36′-0
0.11	.820	5.21	0.11	10 #7 x 13′-9 10 #7 x 12′-9	6	#8 x 50′-6 #7 x 23′-0	2-#3	5 #7 x 24′-0 17 #7 x 21′-6	6	#8 x 47′-9 #7 x 36′-0
0.11	.820	5.74	0.11	10 #7 x 13′-9 15 #7 x 12′-9	6	#8 x 50′-6 #8 x 23′-3	2-#3	5 #7 x 24′-0 20 #7 x 21′-6	6	#8 x 47′-0 #7 x 36′-0
0.10	.806	4.94	0.10	10 #7 x 14′-9 11 #7 x 13′-6	6	#8 x 54′-6 #8 x 26′-3	2-#3	5 #7 x 26′-0 18 #7 x 23′-6	7	#7 x 51′-6 #7 x 39′-0
0.19	.806	6.23	0.19	11 #7 x 14′-9 13 #7 x 13′-6	6	#9 x 54′-6 #8 x 26′-3	4-#3	5 #7 x 26′-0 20 #7 x 23′-6	7	#8 x 51′-6 #7 x 39′-0
0.33	.806	6.55	0.33	13 #7 x 14′-9 15 #7 x 13′-6	6	#9 x 54′-6 #9 x 26′-3	4-#4	6 #7 x 26′-0 25 #7 x 23′-6	7	#8 x 51′-6 #8 x 39′-0

‡ Average cu ft of concrete per sq ft of floor includes ribs, top slabs and drop panels but not column caps.

§ Bars over 60′-0″ long may not be in stock and are difficult to ship, so a lapped splice may be desirable.

When top bars are #7 or larger, use shallow domes to provide cover (page 420).

SLABS ON GROUND *

For any slab on the ground, adequate preparation of subgrade for drainage and compaction is of prime importance. Dowelled expansion joints and weakened plane contraction joints should be carefully located, including expansion joints at all walls.

The design of slabs on the ground to distribute concentrated or uniform loads involves the elastic properties of the subsoil and the slab itself. An analysis can be made but is quite involved. Slabs for the very lightest occupancy should be not less than 4″ thick, and slabs for other occupancies may be empirically selected, the following being about minimum:—

Occupancy **	Min. Slab Thickness	Reinforcement ‡
Sub-slabs under other slabs	2″	None
Domestic or light commercial (loaded less than 100 psf)	4″	One layer 6 x 6 10/10 welded wire fabric, minimum for ideal conditions; 6 x 6 8/8 for average conditions.
Commercial—institutional—barns (loaded 100-200 psf)	5″	One layer 6 x 6 8/8 welded wire fabric or one layer 6 x 6 6/6.
Industrial (loaded not over 400-500 psf) and pavements for industrial plants, gas stations, and garages	6″	One layer 6 x 6 6/6 welded wire fabric or one layer 6 x 6 4/4.
Industrial (loaded 600–800 psf) and heavy pavements for industrial plants, gas stations, and garages	6″	Two layers 6 x 6 6/6 welded wire fabric or two layers 6 x 6 4/4
Industrial (loaded 2000 psf) †	7″	Two mats of bars (one top, one bottom), each of #4 bars @ 12″ c/c, each way
Industrial (loaded 3000 psf) †	8″	Two mats of bars (one top, one bottom), each of #5 bars @ 12″ c/c, each way
Industrial (loaded 4000–5000 psf) †	9″	Two mats of bars (one top, one bottom), each of #5 bars @ 12″ c/c, each way

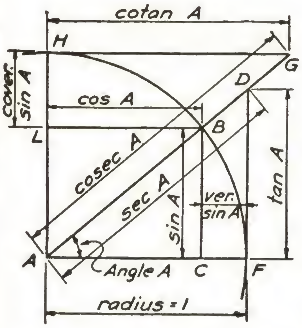
* For further details, see "Concrete Floors on Ground," and "Concrete Airport Pavement," Portland Cement Association, 33 West Grand Avenue, Chicago, Illinois, 1952.

** For loads in excess of, say, 500 psf, use at least 3000 psi quality controlled concrete.

† For loads in excess of, say, 1500 psf the subsoil conditions should be investigated with extra care.

‡ Place first layer of reinforcement 2 in. below top of slab; second layer, 2 in. up from bottom of slab.

TRIGONOMETRIC FORMULAS



Radius, $1 = \sin^2 A + \cos^2 A$

$= \sin A \operatorname{cosec} A = \cos A \sec A = \tan A \cot A$

Sine $A \quad = \dfrac{\cos A}{\cot A} = \dfrac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1 - \cos^2 A} = BC$

Cosine $A \quad = \dfrac{\sin A}{\tan A} = \dfrac{1}{\sec A} = \sin A \cot A = \sqrt{1 - \sin^2 A} = AC$

Tangent $A \quad = \dfrac{\sin A}{\cos A} = \dfrac{1}{\cot A} = \sin A \sec A = FD$

Cotangent $A = \dfrac{\cos A}{\sin A} = \dfrac{1}{\tan A} = \cos A \operatorname{cosec} A = GH$

Secant $A \quad = \dfrac{\tan A}{\sin A} = \dfrac{1}{\cos A} = AD$

Cosecant $A \quad = \dfrac{\cot A}{\cos A} = \dfrac{1}{\sin A} = AG$

$\sin(A \pm B) = \sin A \cos B \pm \cos A \sin B$

$\tan(A \pm B) = \dfrac{\tan A \pm \tan B}{1 \mp \tan A \tan B}$

$\cos(A \pm B) = \cos A \cos B \mp \sin A \sin B$

$\cot(A \pm B) = \dfrac{\cot A \cot B \mp 1}{\cot B \pm \cot A}$

$\sin A + \sin B = 2 \sin \tfrac{1}{2}(A + B) \cos \tfrac{1}{2}(A - B)$

$\tan A + \tan B = \dfrac{\sin(A + B)}{\cos A \cos B}$

$\sin A - \sin B = 2 \cos \tfrac{1}{2}(A + B) \sin \tfrac{1}{2}(A - B)$

$\tan A - \tan B = \dfrac{\sin(A - B)}{\cos A \cos B}$

$\cos A + \cos B = 2 \cos \tfrac{1}{2}(A + B) \cos \tfrac{1}{2}(A - B)$

$\cot A + \cot B = \dfrac{\sin(B + A)}{\sin A \sin B}$

$\cos B - \cos A = 2 \sin \tfrac{1}{2}(A + B) \sin \tfrac{1}{2}(A - B)$

$\cot A - \cot B = \dfrac{\sin(B - A)}{\sin A \sin B}$

$\sin 2A = 2 \sin A \cos A$

$\tan 2A = \dfrac{2 \tan A}{1 - \tan^2 A}$

$\cos 2A = \cos^2 A - \sin^2 A$

$\cot 2A = \dfrac{\cot^2 A - 1}{2 \cot A}$

$\sin \tfrac{1}{2}A = \sqrt{\dfrac{1 - \cos A}{2}} \qquad \cos \tfrac{1}{2}A = \sqrt{\dfrac{1 + \cos A}{2}} \qquad \tan \tfrac{1}{2}A = \dfrac{\sin A}{1 + \cos A} \qquad \cot \tfrac{1}{2}A = \dfrac{\sin A}{1 - \cos A}$

$\sin^2 A = \dfrac{1 - \cos 2A}{2} \qquad \cos^2 A = \dfrac{1 + \cos 2A}{2} \qquad \tan^2 A = \dfrac{1 - \cos 2A}{1 + \cos 2A} \qquad \cot^2 A = \dfrac{1 + \cos 2A}{1 - \cos 2A}$

$\sin^2 A - \sin^2 B = \sin(A + B) \sin(A - B)$

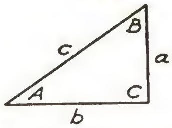
$\cos^2 A - \sin^2 B = \cos(A + B) \cos(A - B)$

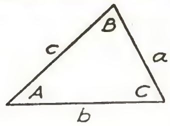
$\dfrac{\sin A \pm \sin B}{\cos A + \cos B} = \tan \tfrac{1}{2}(A \pm B)$

$\dfrac{\sin A \pm \sin B}{\cos B - \cos A} = \cot \tfrac{1}{2}(A \mp B)$

Quadrant	I	II	III	IV	Angle			Angle $a < 90°$				
Angles	0° to 90°	90° to 180°	180° to 270°	270° to 360°	30°	45°	60°	Angle	sin	cos	tan	cot
Functions	Values vary from				Equivalent values			$\phi°$	$\phi°$	$\phi°$	$\phi°$	$\phi°$
sin	$+0$ to $+1$	$+1$ to $+0$	-0 to -1	-1 to -0	$\tfrac{1}{2}$	$\tfrac{1}{2}\sqrt{2}$	$\tfrac{1}{2}\sqrt{3}$	$0° \pm a$	$\pm\sin a$	$+\cos a$	$\pm\tan a$	$\pm\cot a$
cos	$+1$ to $+0$	-0 to -1	-1 to -0	$+0$ to $+1$	$\tfrac{1}{2}\sqrt{3}$	$\tfrac{1}{2}\sqrt{2}$	$\tfrac{1}{2}$	$90° \pm a$	$+\cos a$	$\mp\sin a$	$\mp\cot a$	$\mp\tan a$
tan	$+0$ to $+\infty$	$-\infty$ to -0	$+0$ to $+\infty$	$-\infty$ to -0	$\tfrac{1}{3}\sqrt{3}$	1	$\sqrt{3}$	$180° \pm a$	$\mp\sin a$	$-\cos a$	$\pm\tan a$	$\pm\cot a$
cot	$+\infty$ to $+0$	-0 to $-\infty$	$+\infty$ to $+0$	-0 to $-\infty$	$\sqrt{3}$	1	$\tfrac{1}{3}\sqrt{3}$	$270° \pm a$	$-\cos a$	$\pm\sin a$	$\mp\cot a$	$\mp\tan a$

TRIGONOMETRIC SOLUTION OF TRIANGLES





$$S = \frac{a+b+c}{2}$$

Given	Sought	Formulas
		Right-Angled Triangles
a, c	A, B, b	$\sin A = \frac{a}{c}$, $\cos B = \frac{a}{c}$, $b = \sqrt{c^2 - a^2}$
	Area	Area $= \frac{a}{2}\sqrt{c^2 - a^2}$
a, b	A, B, c	$\tan A = \frac{a}{b}$, $\tan B = \frac{b}{a}$, $c = \sqrt{a^2 + b^2}$
	Area	Area $= \frac{ab}{2}$
A, a	B, b, c	$B = 90° - A$, $b = a \cot A$, $c = \frac{a}{\sin A}$
	Area	Area $= \frac{a^2 \cot A}{2}$
A, b	B, a, c	$B = 90° - A$, $a = b \tan A$, $c = \frac{b}{\cos A}$
	Area	Area $= \frac{b^2 \tan A}{2}$
A, c	B, a, b	$B = 90° - A$, $a = c \sin A$, $b = c \cos A$
	Area	Area $= \frac{c^2 \sin A \cos A}{2} = \frac{c^2 \sin 2A}{4}$
		Oblique-Angled Triangles
a, b, c	A	$\sin \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{bc}}$, $\cos \frac{1}{2} A = \sqrt{\frac{s(s-a)}{bc}}$, $\tan \frac{1}{2} A = \sqrt{\frac{(s-b)(s-c)}{s(s-a)}}$
	B	$\sin \frac{1}{2} B = \sqrt{\frac{(s-a)(s-c)}{ac}}$, $\cos \frac{1}{2} B = \sqrt{\frac{s(s-b)}{ac}}$, $\tan \frac{1}{2} B = \sqrt{\frac{(s-a)(s-c)}{s(s-b)}}$
	C	$\sin \frac{1}{2} C = \sqrt{\frac{(s-a)(s-b)}{ab}}$, $\cos \frac{1}{2} C = \sqrt{\frac{s(s-c)}{ab}}$, $\tan \frac{1}{2} C = \sqrt{\frac{(s-a)(s-b)}{s(s-c)}}$
	Area	Area $= \sqrt{s(s-a)(s-b)(s-c)}$
a, A, B	b, c	$b = \frac{a \sin B}{\sin A}$, $c = \frac{a \sin C}{\sin A} = \frac{a \sin (A+B)}{\sin A}$
	Area	Area $= \frac{1}{2} ab \sin C = \frac{a^2 \sin B \sin C}{2 \sin A}$
a, b, A	B	$\sin B = \frac{b \sin A}{a}$
	c	$c = \frac{a \sin C}{\sin A} = \frac{b \sin C}{\sin B} = \sqrt{a^2 + b^2 - 2ab \cos C}$
	Area	Area $= \frac{1}{2} ab \sin C$
a, b, C	A	$\tan A = \frac{a \sin C}{b - a \cos C}$, $\tan \frac{1}{2}(A - B) = \frac{a-b}{a+b} \cot \frac{1}{2} C$
	c	$c = \sqrt{a^2 + b^2 - 2ab \cos C} = \frac{a \sin C}{\sin A}$
	Area	Area $= \frac{1}{2} ab \sin C$

$a^2 = b^2 + c^2 - 2bc \cos A$, $b^2 = a^2 + c^2 - 2ac \cos B$, $c^2 = a^2 + b^2 - 2ab \cos C$

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	SINES							Cosines
	0′	10′	20′	30′	40′	50′	60′	
0	0.00000	0.00291	0.00582	0.00873	0.01164	0.01454	0.01745	89
1	0.01745	0.02036	0.02327	0.02618	0.02908	0.03199	0.03490	88
2	0.03490	0.03781	0.04071	0.04362	0.04653	0.04943	0.05234	87
3	0.05234	0.05524	0.05814	0.06105	0.06395	0.06685	0.06976	86
4	0.06976	0.07266	0.07556	0.07846	0.08136	0.08426	0.08716	85
5	0.08716	0.09005	0.09295	0.09585	0.09874	0.10164	0.10453	84
6	0.10453	0.10742	0.11031	0.11320	0.11609	0.11898	0.12187	83
7	0.12187	0.12476	0.12764	0.13053	0.13341	0.13629	0.13917	82
8	0.13917	0.14205	0.14493	0.14781	0.15069	0.15356	0.15643	81
9	0.15643	0.15931	0.16218	0.16505	0.16792	0.17078	0.17365	80
10	0.17365	0.17651	0.17937	0.18224	0.18509	0.18795	0.19081	79
11	0.19081	0.19366	0.19652	0.19937	0.20222	0.20507	0.20791	78
12	0.20791	0.21076	0.21360	0.21644	0.21928	0.22212	0.22495	77
13	0.22495	0.22778	0.23062	0.23345	0.23627	0.23910	0.24192	76
14	0.24192	0.24474	0.24756	0.25038	0.25320	0.25601	0.25882	75
15	0.25882	0.26163	0.26443	0.26724	0.27004	0.27284	0.27564	74
16	0.27564	0.27843	0.28123	0.28402	0.28680	0.28959	0.29237	73
17	0.29237	0.29515	0.29793	0.30071	0.30348	0.30625	0.30902	72
18	0.30902	0.31178	0.31454	0.31730	0.32006	0.32282	0.32557	71
19	0.32557	0.32832	0.33106	0.33381	0.33655	0.33929	0.34202	70
20	0.34202	0.34475	0.34748	0.35021	0.35293	0.35565	0.35837	69
21	0.35837	0.36108	0.36379	0.36650	0.36921	0.37191	0.37461	68
22	0.37461	0.37730	0.37999	0.38268	0.38537	0.38805	0.39073	67
23	0.39073	0.39341	0.39608	0.39875	0.40142	0.40408	0.40674	66
24	0.40674	0.40939	0.41204	0.41469	0.41734	0.41998	0.42262	65
25	0.42262	0.42525	0.42788	0.43051	0.43313	0.43575	0.43837	64
26	0.43837	0.44098	0.44359	0.44620	0.44880	0.45140	0.45399	63
27	0.45399	0.45658	0.45917	0.46175	0.46433	0.46690	0.46947	62
28	0.46947	0.47204	0.47460	0.47716	0.47971	0.48226	0.48481	61
29	0.48481	0.48735	0.48989	0.49242	0.49495	0.49748	0.50000	60
30	0.50000	0.50252	0.50503	0.50754	0.51004	0.51254	0.51504	59
31	0.51504	0.51753	0.52002	0.52250	0.52498	0.52745	0.52992	58
32	0.52992	0.53238	0.53484	0.53730	0.53975	0.54220	0.54464	57
33	0.54464	0.54708	0.54951	0.55194	0.55436	0.55678	0.55919	56
34	0.55919	0.56160	0.56401	0.56641	0.56880	0.57119	0.57358	55
35	0.57358	0.57596	0.57833	0.58070	0.58307	0.58543	0.58779	54
36	0.58779	0.59014	0.59248	0.59482	0.59716	0.59949	0.60182	53
37	0.60182	0.60414	0.60645	0.60876	0.61107	0.61337	0.61566	52
38	0.61566	0.61795	0.62024	0.62251	0.62479	0.62706	0.62932	51
39	0.62932	0.63158	0.63383	0.63608	0.63832	0.64056	0.64279	50
40	0.64279	0.64501	0.64723	0.64945	0.65166	0.65386	0.65606	49
41	0.65606	0.65825	0.66044	0.66262	0.66480	0.66697	0.66913	48
42	0.66913	0.67129	0.67344	0.67559	0.67773	0.67987	0.68200	47
43	0.68200	0.68412	0.68624	0.68835	0.69046	0.69256	0.69466	46
44	0.69466	0.69675	0.69883	0.70091	0.70298	0.70505	0.70711	45
Sines	60′	50′	40′	30′	20′	10′	0′	Degrees
	COSINES							

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	COSINES							Sines
	0′	10′	20′	30′	40′	50′	60′	
0	1.00000	1.00000	0.99998	0.99996	0.99993	0.99989	0.99985	89
1	0.99985	0.99979	0.99973	0.99966	0.99958	0.99949	0.99939	88
2	0.99939	0.99929	0.99917	0.99905	0.99892	0.99878	0.99863	87
3	0.99863	0.99847	0.99831	0.99813	0.99795	0.99776	0.99756	86
4	0.99756	0.99736	0.99714	0.99692	0.99668	0.99644	0.99619	85
5	0.99619	0.99594	0.99567	0.99540	0.99511	0.99482	0.99452	84
6	0.99452	0.99421	0.99390	0.99357	0.99324	0.99290	0.99255	83
7	0.99255	0.99219	0.99182	0.99144	0.99106	0.99067	0.99027	82
8	0.99027	0.98986	0.98944	0.98902	0.98858	0.98814	0.98769	81
9	0.98769	0.98723	0.98676	0.98629	0.98580	0.98531	0.98481	80
10	0.98481	0.98430	0.98378	0.98325	0.98272	0.98218	0.98163	79
11	0.98163	0.98107	0.98050	0.97992	0.97934	0.97875	0.97815	78
12	0.97815	0.97754	0.97692	0.97630	0.97566	0.97502	0.97437	77
13	0.97437	0.97371	0.97304	0.97237	0.97169	0.97100	0.97030	76
14	0.97030	0.96959	0.96887	0.96815	0.96742	0.96667	0.96593	75
15	0.96593	0.96517	0.96440	0.96363	0.96285	0.96206	0.96126	74
16	0.96126	0.96046	0.95964	0.95882	0.95799	0.95715	0.95630	73
17	0.95630	0.95545	0.95459	0.95372	0.95284	0.95195	0.95106	72
18	0.95106	0.95015	0.94924	0.94832	0.94740	0.94646	0.94552	71
19	0.94552	0.94457	0.94361	0.94264	0.94167	0.94068	0.93969	70
20	0.93969	0.93869	0.93769	0.93667	0.93565	0.93462	0.93358	69
21	0.93358	0.93253	0.93148	0.93042	0.92935	0.92827	0.92718	68
22	0.92718	0.92609	0.92499	0.92388	0.92276	0.92164	0.92050	67
23	0.92050	0.91936	0.91822	0.91706	0.91590	0.91472	0.91355	66
24	0.91355	0.91236	0.91116	0.90996	0.90875	0.90753	0.90631	65
25	0.90631	0.90507	0.90383	0.90259	0.90133	0.90007	0.89879	64
26	0.89879	0.89752	0.89623	0.89493	0.89363	0.89232	0.89101	63
27	0.89101	0.88968	0.88835	0.88701	0.88566	0.88431	0.88295	62
28	0.88295	0.88158	0.88020	0.87882	0.87743	0.87603	0.87462	61
29	0.87462	0.87321	0.87178	0.87036	0.86892	0.86748	0.86603	60
30	0.86603	0.86457	0.86310	0.86163	0.86015	0.85866	0.85717	59
31	0.85717	0.85567	0.85416	0.85264	0.85112	0.84959	0.84805	58
32	0.84805	0.84650	0.84495	0.84339	0.84182	0.84025	0.83867	57
33	0.83867	0.83708	0.83549	0.83389	0.83228	0.83066	0.82904	56
34	0.82904	0.82741	0.82577	0.82413	0.82248	0.82082	0.81915	55
35	0.81915	0.81748	0.81580	0.81412	0.81242	0.81072	0.80902	54
36	0.80902	0.80730	0.80558	0.80386	0.80212	0.80038	0.79864	53
37	0.79864	0.79688	0.79512	0.79335	0.79158	0.78980	0.78801	52
38	0.78801	0.78622	0.78442	0.78261	0.78079	0.77897	0.77715	51
39	0.77715	0.77531	0.77347	0.77162	0.76977	0.76791	0.76604	50
40	0.76604	0.76417	0.76229	0.76041	0.75851	0.75661	0.75471	49
41	0.75471	0.75280	0.75088	0.74896	0.74703	0.74509	0.74314	48
42	0.74314	0.74120	0.73924	0.73728	0.73531	0.73333	0.73135	47
43	0.73135	0.72937	0.72737	0.72537	0.72337	0.72136	0.71934	46
44	0.71934	0.71732	0.71529	0.71325	0.71121	0.70916	0.70711	45
Cosines	60′	50′	40′	30′	20′	10′	0′	Degrees
	SINES							

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	TANGENTS 0′	10′	20′	30′	40′	50′	60′	Cotangents
0	0.00000	0.00291	0.00582	0.00873	0.01164	0.01455	0.01746	89
1	0.01746	0.02036	0.02328	0.02619	0.02910	0.03201	0.03492	88
2	0.03492	0.03783	0.04075	0.04366	0.04658	0.04949	0.05241	87
3	0.05241	0.05533	0.05824	0.06116	0.06408	0.06700	0.06993	86
4	0.06993	0.07285	0.07578	0.07870	0.08163	0.08456	0.08749	85
5	0.08749	0.09042	0.09335	0.09629	0.09923	0 10216	0.10510	84
6	0.10510	0.10805	0.11099	0.11394	0.11688	0.11983	0.12278	83
7	0.12278	0.12574	0.12869	0.13165	0.13461	0.13758	0.14054	82
8	0.14054	0.14351	0.14648	0.14945	0.15243	0.15540	0.15838	81
9	0.15838	0.16137	0.16435	0.16734	0.17033	0.17333	0.17633	80
10	0.17633	0.17933	0.18233	0.18534	0.18835	0.19136	0.19438	79
11	0.19438	0.19740	0.20042	0.20345	0.20648	0.20952	0.21256	78
12	0.21256	0.21560	0.21864	0.22169	0.22475	0.22781	0.23087	77
13	0.23087	0.23393	0.23700	0.24008	0.24316	0.24624	0.24933	76
14	0.24933	0.25242	0.25552	0.25862	0.26172	0.26483	0.26795	75
15	0.26795	0.27107	0.27419	0.27732	0.28046	0.28360	0.28675	74
16	0.28675	0.28990	0.29305	0.29621	0.29938	0.30255	0.30573	73
17	0.30573	0.30891	0.31210	0.31530	0.31850	0.32171	0.32492	72
18	0.32492	0.32814	0.33136	0.33460	0.33783	0.34108	0.34433	71
19	0.34433	0.34758	0.35085	0.35412	0.35740	0.36068	0.36397	70
20	0.36397	0.36727	0.37057	0.37388	0.37720	0.38053	0.38386	69
21	0.38386	0.38721	0.39055	0.39391	0.39727	0.40065	0.40403	68
22	0.40403	0.40741	0.41081	0.41421	0.41763	0.42105	0.42447	67
23	0.42447	0.42791	0.43136	0.43481	0.43828	0.44175	0.44523	66
24	0.44523	0.44872	0.45222	0.45573	0.45924	0.46277	0.46631	65
25	0.46631	0.46985	0.47341	0.47698	0.48055	0.48414	0.48773	64
26	0.48773	0.49134	0.49495	0.49858	0.50222	0.50587	0.50953	63
27	0.50953	0.51320	0.51688	0.52057	0.52427	0.52798	0.53171	62
28	0.53171	0.53545	0.53920	0.54296	0.54674	0.55051	0.55431	61
29	0.55431	0.55812	0.56194	0.56577	0.56962	0.57348	0.57735	60
30	0.57735	0.58124	0.58513	0.58905	0.59297	0.59691	0.60086	59
31	0.60086	0.60483	0.60881	0.61280	0.61681	0.62083	0.62487	58
32	0.62487	0.62892	0.63299	0.63707	0.64117	0.64528	0.64941	57
33	0.64941	0.65355	0.65771	0.66189	0.66608	0.67028	0.67451	56
34	0.67451	0.67875	0.68301	0.68728	0.69157	0.69588	0.70021	55
35	0.70021	0.70455	0.70891	0.71329	0.71769	0.72211	0.72654	54
36	0.72654	0.73100	0.73547	0.73996	0.74447	0.74900	0.75355	53
37	0.75355	0.75812	0.76272	0.76733	0.77196	0.77661	0.78129	52
38	0.78129	0.78598	0.79070	0.79544	0.80020	0.80498	0.80978	51
39	0.80978	0.81461	0.81946	0.82434	0.82923	0.83415	0.83910	50
40	0.83910	0.84407	0.84906	0.85408	0.85912	0.86419	0.86929	49
41	0.86929	0.87441	0.87955	0.88473	0.88992	0.89515	0.90040	48
42	0.90040	0.90569	0.91099	0.91633	0.92170	0.92709	0.93252	47
43	0.93252	0.93797	0.94345	0.94896	0.95451	0.96008	0.96569	46
44	0.96569	0.97133	0.97700	0.98270	0.98843	0.99420	1.00000	45
Tangents	60′	50′	40′	30′	20′	10′	0′	Degrees
	COTANGENTS							

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	COTANGENTS							Tangents
	0′	10′	20′	30′	40′	50′	60′	
0	∞	343.77371	171.88540	114.58865	85.93979	68.75009	57.28996	89
1	57.28996	49.10388	42.96408	38.18846	34.36777	31.24158	28.63625	88
2	28.63625	26.43160	24.54176	22.90377	21.47040	20.20555	19.08114	87
3	19.08114	18.07498	17.16934	16.34986	15.60478	14.92442	14.30067	86
4	14.30067	13.72674	13.19688	12.70621	12.25051	11.82617	11.43005	85
5	11.43005	11.05943	10.71191	10.38540	10.07803	9.78817	9.51436	84
6	9.51436	9.25530	9.00983	8.77689	8.55555	8.34496	8.14435	83
7	8.14435	7.95302	7.77035	7.59575	7.42871	7.26873	7.11537	82
8	7.11537	6.96823	6.82694	6.69116	6.56055	6.43484	6.31375	81
9	6.31375	6.19703	6.08444	5.97576	5.87080	5.76937	5.67128	80
10	5.67128	5.57638	5.48451	5.39552	5.30928	5.22566	5.14455	79
11	5.14455	5.06584	4.98940	4.91516	4.84300	4.77286	4.70463	78
12	4.70463	4.63825	4.57363	4.51071	4.44942	4.38969	4.33148	77
13	4.33148	4.27471	4.21933	4.16530	4.11256	4.06107	4.01078	76
14	4.01078	3.96165	3.91364	3.86671	3.82083	3.77595	3.73205	75
15	3.73205	3.68909	3.64705	3.60588	3.56557	3.52609	3.48741	74
16	3.48741	3.44951	3.41236	3.37594	3.34023	3.30521	3.27085	73
17	3.27085	3.23714	3.20406	3.17159	3.13972	3.10842	3.07768	72
18	3.07768	3.04749	3.01783	2.98869	2.96004	2.93189	2.90421	71
19	2.90421	2.87700	2.85023	2.82391	2.79802	2.77254	2.74748	70
20	2.74748	2.72281	2.69853	2.67462	2.65109	2.62791	2.60509	69
21	2.60509	2.58261	2.56046	2.53865	2.51715	2.49597	2.47509	68
22	2.47509	2.45451	2.43422	2.41421	2.39449	2.37504	2.35585	67
23	2.35585	2.33693	2.31826	2.29984	2.28167	2.26374	2.24604	66
24	2.24604	2.22857	2.21132	2.19430	2.17749	2.16090	2.14451	65
25	2.14451	2.12832	2.11233	2.09654	2.08094	2.06553	2.05030	64
26	2.05030	2.03526	2.02039	2.00569	1.99116	1.97680	1.96261	63
27	1.96261	1.94858	1.93470	1.92098	1.90741	1.89400	1.88073	62
28	1.88073	1.86760	1.85462	1.84177	1.82907	1.81649	1.80405	61
29	1.80405	1.79174	1.77955	1.76749	1.75556	1.74375	1.73205	60
30	1.73205	1.72047	1.70901	1.69766	1.68643	1.67530	1.66428	59
31	1.66428	1.65337	1.64256	1.63185	1.62125	1.61074	1.60033	58
32	1.60033	1.59002	1.57981	1.56969	1.55966	1.54972	1.53987	57
33	1.53987	1.53010	1.52043	1.51084	1.50133	1.49190	1.48256	56
34	1.48256	1.47330	1.46411	1.45501	1.44598	1.43703	1.42815	55
35	1.42815	1.41934	1.41061	1.40195	1.39336	1.38484	1.37638	54
36	1.37638	1.36800	1.35968	1.35142	1.34323	1.33511	1.32704	53
37	1.32704	1.31904	1.31110	1.30323	1.29541	1.28764	1.27994	52
38	1.27994	1.27230	1.26471	1.25717	1.24969	1.24227	1.23490	51
39	1.23490	1.22758	1.22031	1.21310	1.20593	1.19882	1.19175	50
40	1.19175	1.18474	1.17777	1.17085	1.16398	1.15715	1.15037	49
41	1.15037	1.14363	1.13694	1.13029	1.12369	1.11713	1.11061	48
42	1.11061	1.10414	1.09770	1.09131	1.08496	1.07864	1.07237	47
43	1.07237	1.06613	1.05994	1.05378	1.04766	1.04158	1.03553	46
44	1.03553	1.02952	1.02355	1.01761	1.01170	1.00583	1.00000	45
Cotangents	60′	50′	40′	30′	20′	10′	0′	Degrees
	TANGENTS							

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	SECANTS							Cosecants
	0′	10′	20′	30′	40′	50′	60′	
0	1.00000	1.00000	1.00002	1.00004	1.00007	1.00011	1.00015	89
1	1.00015	1.00021	1.00027	1.00034	1.00042	1.00051	1.00061	88
2	1.00061	1.00072	1.00083	1.00095	1.00108	1.00122	1.00137	87
3	1.00137	1.00153	1.00169	1.00187	1.00205	1.00224	1.00244	86
4	1.00244	1.00265	1.00287	1.00309	1.00333	1.00357	1.00382	85
5	1.00382	1.00408	1.00435	1.00463	1.00491	1.00521	1.00551	84
6	1.00551	1.00582	1.00614	1.00647	1.00681	1.00715	1.00751	83
7	1.00751	1.00787	1.00825	1.00863	1.00902	1.00942	1.00983	82
8	1.00983	1.01024	1.01067	1.01111	1.01155	1.01200	1.01247	81
9	1.01247	1.01294	1.01342	1.01391	1.01440	1.01491	1.01543	80
10	1.01543	1.01595	1.01649	1.01703	1.01758	1.01815	1.01872	79
11	1.01872	1.01930	1.01989	1.02049	1.02110	1.02171	1.02234	78
12	1.02234	1.02298	1.02362	1.02428	1.02494	1.02562	1.02630	77
13	1.02630	1.02700	1.02770	1.02842	1.02914	1.02987	1.03061	76
14	1.03061	1.03137	1.03213	1.03290	1.03368	1.03447	1.03528	75
15	1.03528	1.03609	1.03691	1.03774	1.03858	1.03944	1.04030	74
16	1.04030	1.04117	1.04206	1.04295	1.04385	1.04477	1.04569	73
17	1.04569	1.04663	1.04757	1.04853	1.04950	1.05047	1.05146	72
18	1.05146	1.05246	1.05347	1.05449	1.05552	1.05657	1.05762	71
19	1.05762	1.05869	1.05976	1.06085	1.06195	1.06306	1.06418	70
20	1.06418	1.06531	1.06645	1.06761	1.06878	1.06995	1.07115	69
21	1.07115	1.07235	1.07356	1.07479	1.07602	1.07727	1.07853	68
22	1.07853	1.07981	1.08109	1.08239	1.08370	1.08503	1.08636	67
23	1.08636	1.08771	1.08907	1.09044	1.09183	1.09323	1.09464	66
24	1.09464	1.09606	1.09750	1.09895	1.10041	1.10189	1.10338	65
25	1.10338	1.10488	1.10640	1.10793	1.10947	1.11103	1.11260	64
26	1.11260	1.11419	1.11579	1.11740	1.11903	1.12067	1.12233	63
27	1.12233	1.12400	1.12568	1.12738	1.12910	1.13083	1.13257	62
28	1.13257	1.13433	1.13610	1.13789	1.13970	1.14152	1.14335	61
29	1.14335	1.14521	1.14707	1.14896	1.15085	1.15277	1.15470	60
30	1.15470	1.15665	1.15861	1.16059	1.16259	1.16460	1.16663	59
31	1.16663	1.16868	1.17075	1.17283	1.17493	1.17704	1.17918	58
32	1.17918	1.18133	1.18350	1.18569	1.18790	1.19012	1.19236	57
33	1.19236	1.19463	1.19691	1.19920	1.20152	1.20386	1.20622	56
34	1.20622	1.20859	1.21099	1.21341	1.21584	1.21830	1.22077	55
35	1.22077	1.22327	1.22579	1.22833	1.23089	1.23347	1.23607	54
36	1.23607	1.23869	1.24134	1.24400	1.24669	1.24940	1.25214	53
37	1.25214	1.25489	1.25767	1.26047	1.26330	1.26615	1.26902	52
38	1.26902	1.27191	1.27483	1.27778	1.28075	1.28374	1.28676	51
39	1.28676	1.28980	1.29287	1.29597	1.29909	1.30223	1.30541	50
40	1.30541	1.30861	1.31183	1.31509	1.31837	1.32168	1.32501	49
41	1.32501	1.32838	1.33177	1.33519	1.33864	1.34212	1.34563	48
42	1.34563	1.34917	1.35274	1.35634	1.35997	1.36363	1.36733	47
43	1.36733	1.37105	1.37481	1.37860	1.38242	1.38628	1.39016	46
44	1.39016	1.39409	1.39804	1.40203	1.40606	1.41012	1.41421	45
Secants	60′	50′	40′	30′	20′	10′	0′	Degrees
	COSECANTS							

NATURAL TRIGONOMETRIC FUNCTIONS

Degrees	COSECANTS 0′	10′	20′	30′	40′	50′	60′	Secants
0	∞	343.77516	171.88831	114.59301	85.94561	68.75736	57.29869	89
1	57.29869	49.11406	42.97571	38.20155	34.38232	31.25758	28.65371	88
2	28.65371	26.45051	24.56212	22.92559	21.49368	20.23028	19.10732	87
3	19.10732	18.10262	17.19843	16.38041	15.63679	14.95788	14.33559	86
4	14.33559	13.76312	13.23472	12.74550	12.29125	11.86837	11.47371	85
5	11.47371	11.10455	10.75849	10.43343	10.12752	9.83912	9.56677	84
6	9.56677	9.30917	9.06515	8.83367	8.61379	8.40466	8.20551	83
7	8.20551	8.01565	7.83443	7.66130	7.49571	7.33719	7.18530	82
8	7.18530	7.03962	6.89979	6.76547	6.63633	6.51208	6.39245	81
9	6.39245	6.27719	6.16607	6.05886	5.95536	5.85539	5.75877	80
10	5.75877	5.66533	5.57493	5.48740	5.40263	5.32049	5.24084	79
11	5.24084	5.16359	5.08863	5.01585	4.94517	4.87649	4.80973	78
12	4.80973	4.74482	4.68167	4.62023	4.56041	4.50216	4.44541	77
13	4.44541	4.39012	4.33622	4.28366	4.23239	4.18238	4.13357	76
14	4.13357	4.08591	4.03938	3.99393	3.94952	3.90613	3.86370	75
15	3.86370	3.82223	3.78166	3.74198	3.70315	3.66515	3.62796	74
16	3.62796	3.59154	3.55587	3.52094	3.48671	3.45317	3.42030	73
17	3.42030	3.38808	3.35649	3.32551	3.29512	3.26531	3.23607	72
18	3.23607	3.20737	3.17920	3.15155	3.12440	3.09774	3.07155	71
19	3.07155	3.04584	3.02057	2.99574	2.97135	2.94737	2.92380	70
20	2.92380	2.90063	2.87785	2.85545	2.83342	2.81175	2.79043	69
21	2.79043	2.76945	2.74881	2.72850	2.70851	2.68884	2.66947	68
22	2.66947	2.65040	2.63162	2.61313	2.59491	2.57698	2.55930	67
23	2.55930	2.54190	2.52474	2.50784	2.49119	2.47477	2.45859	66
24	2.45859	2.44264	2.42692	2.41142	2.39614	2.38107	2.36620	65
25	2.36620	2.35154	2.33708	2.32282	2.30875	2.29487	2.28117	64
26	2.28117	2.26766	2.25432	2.24116	2.22817	2.21535	2.20269	63
27	2.20269	2.19019	2.17786	2.16568	2.15366	2.14178	2.13005	62
28	2.13005	2.11847	2.10704	2.09574	2.08458	2.07356	2.06267	61
29	2.06267	2.05191	2.04128	2.03077	2.02039	2.01014	2.00000	60
30	2.00000	1.98998	1.98008	1.97029	1.96062	1.95106	1.94160	59
31	1.94160	1.93226	1.92302	1.91388	1.90485	1.89591	1.88709	58
32	1.88708	1.87834	1.86970	1.86116	1.85271	1.84435	1.83608	57
33	1.83608	1.82790	1.81981	1.81180	1.80388	1.79604	1.78829	56
34	1.78829	1.78062	1.77303	1.76552	1.75808	1.75073	1.74345	55
35	1.74345	1.73624	1.72911	1.72205	1.71506	1.70815	1.70130	54
36	1.70130	1.69452	1.68782	1.68117	1.67460	1.66809	1.66164	53
37	1.66164	1.65526	1.64894	1.64268	1.63648	1.63035	1.62427	52
38	1.62427	1.61825	1.61229	1.60639	1.60054	1.59475	1.58902	51
39	1.58902	1.58333	1.57771	1.57213	1.56661	1.56114	1.55572	50
40	1.55572	1.55036	1.54504	1.53977	1.53455	1.52938	1.52425	49
41	1.52425	1.51918	1.51415	1.50916	1.50422	1.49933	1.49448	48
42	1.49448	1.48967	1.48491	1.48019	1.47551	1.47087	1.46628	47
43	1.46628	1.46173	1.45721	1.45274	1.44831	1.44391	1.43956	46
44	1.43956	1.43524	1.43096	1.42672	1.42251	1.41835	1.41421	45
Cosecants	60′	50′	40′	30′	20′	10′	0′	Degrees
	SECANTS							

FUNCTIONS OF NUMBERS, 1 to 50

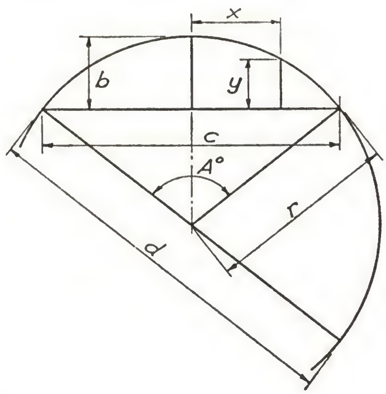
No.	Square	Cube	Square Root	Cube Root	Logarithm	1000 x Reciprocal	No. = Diameter	
							Circum	Area
1	1	1	1.0000	1.0000	0.00000	1000.000	3.142	0.7854
2	4	8	1.4142	1.2599	0.30103	500.000	6.283	3.1416
3	9	27	1.7321	1.4422	0.47712	333.333	9.425	7.0686
4	16	64	2.0000	1.5874	0.60206	250.000	12.566	12.5664
5	25	125	2.2361	1.7100	0.69897	200.000	15.708	19.6350
6	36	216	2.4495	1.8171	0.77815	166.667	18.850	28.2743
7	49	343	2.6458	1.9129	0.84510	142.857	21.991	38.4845
8	64	512	2.8284	2.0000	0.90309	125.000	25.133	50.2655
9	81	729	3.0000	2.0801	0.95424	111.111	28.274	63.6173
10	100	1000	3.1623	2.1544	1.00000	100.000	31.416	78.5398
11	121	1331	3.3166	2.2240	1.04139	90.9091	34.558	95.0332
12	144	1728	3.4641	2.2894	1.07918	83.3333	37.699	113.097
13	169	2197	3.6056	2.3513	1.11394	76.9231	40.841	132.732
14	196	2744	3.7417	2.4101	1.14613	71.4286	43.982	153.938
15	225	3375	3.8730	2.4662	1.17609	66.6667	47.124	176.715
16	256	4096	4.0000	2.5198	1.20412	62.5000	50.265	201.062
17	289	4913	4.1231	2.5713	1.23045	58.8235	53.407	226.980
18	324	5832	4.2426	2.6207	1.25527	55.5556	56.549	254.469
19	361	6859	4.3589	2.6684	1.27875	52.6316	59.690	283.529
20	400	8000	4.4721	2.7144	1.30103	50.0000	62.832	314.159
21	441	9261	4.5826	2.7589	1.32222	47.6190	65.973	346.361
22	484	10648	4.6904	2.8020	1.34242	45.4545	69.115	380.133
23	529	12167	4.7958	2.8439	1.36173	43.4783	72.257	415.476
24	576	13824	4.8990	2.8845	1.38021	41.6667	75.398	452.389
25	625	15625	5.0000	2.9240	1.39794	40.0000	78.540	490.874
26	676	17576	5.0990	2.9625	1.41497	38.4615	81.681	530.929
27	729	19683	5.1962	3.0000	1.43136	37.0370	84.823	572.555
28	784	21952	5.2915	3.0366	1.44716	35.7143	87.965	615.752
29	841	24389	5.3852	3.0723	1.46240	34.4828	91.106	660.520
30	900	27000	5.4772	3.1072	1.47712	33.3333	94.248	706.858
31	961	29791	5.5678	3.1414	1.49136	32.2581	97.389	754.768
32	1024	32768	5.6569	3.1748	1.50515	31.2500	100.53	804.248
33	1089	35937	5.7446	3.2075	1.51851	30.3030	103.67	855.299
34	1156	39304	5.8310	3.2396	1.53148	29.4118	106.81	907.920
35	1225	42875	5.9161	3.2711	1.54407	28.5714	109.96	962.113
36	1296	46656	6.0000	3.3019	1.55630	27.7778	113.10	1017.88
37	1369	50653	6.0828	3.3322	1.56820	27.0270	116.24	1075.21
38	1444	54872	6.1644	3.3620	1.57978	26.3158	119.38	1134.11
39	1521	59319	6.2450	3.3912	1.59106	25.6410	122.52	1194.59
40	1600	64000	6.3246	3.4200	1.60206	25.0000	125.66	1256.64
41	1681	68921	6.4031	3.4482	1.61278	24.3902	128.81	1320.25
42	1764	74088	6.4807	3.4760	1.62325	23.8095	131.95	1385.44
43	1849	79507	6.5574	3.5034	1.63347	23.2558	135.09	1452.20
44	1936	85184	6.6332	3.5303	1.64345	22.7273	138.23	1520.53
45	2025	91125	6.7082	3.5569	1.65321	22.2222	141.37	1590.43
46	2116	97336	6.7823	3.5830	1.66276	21.7391	144.51	1661.90
47	2209	103823	6.8557	3.6088	1.67210	21.2766	147.65	1734.94
48	2304	110592	6.9282	3.6342	1.68124	20.8333	150.80	1809.56
49	2401	117649	7.0000	3.6593	1.69020	20.4082	153.94	1885.74
50	2500	125000	7.0711	3.6840	1.69897	20.0000	157.08	1963.50

FUNCTIONS OF NUMBERS, 51 to 100

No.	Square	Cube	Square Root	Cube Root	Logarithm	1000 x Reciprocal	No. = Diameter	
							Circum	Area
51	2601	132651	7.1414	3.7084	1.70757	19.6078	160.22	2042.82
52	2704	140608	7.2111	3.7325	1.71600	19.2308	163.36	2123.72
53	2809	148877	7.2801	3.7563	1.72428	18.8679	166.50	2206.18
54	2916	157464	7.3485	3.7798	1.73239	18.5185	169.65	2290.22
55	3025	166375	7.4162	3.8030	1.74036	18.1818	172.79	2375.83
56	3136	175616	7.4833	3.8259	1.74819	17.8571	175.93	2463.01
57	3249	185193	7.5498	3.8485	1.75587	17.5439	179.07	2551.76
58	3364	195112	7.6158	3.8709	1.76343	17.2414	182.21	2642.08
59	3481	205379	7.6811	3.8930	1.77085	16.9492	185.35	2733.97
60	3600	216000	7.7460	3.9149	1.77815	16.6667	188.50	2827.43
61	3721	226981	7.8102	3.9365	1.78533	16.3934	191.64	2922.47
62	3844	238328	7.8740	3.9579	1.79239	16.1290	194.78	3019.07
63	3969	250047	7.9373	3.9791	1.79934	15.8730	197.92	3117.25
64	4096	262144	8.0000	4.0000	1.80618	15.6250	201.06	3216.99
65	4225	274625	8.0623	4.0207	1.81291	15.3846	204.20	3318.31
66	4356	287496	8.1240	4.0412	1.81954	15.1515	207.35	3421.19
67	4489	300763	8.1854	4.0615	1.82607	14.9254	210.49	3525.65
68	4624	314432	8.2462	4.0817	1.83251	14.7059	213.63	3631.68
69	4761	328509	8.3066	4.1016	1.83885	14.4928	216.77	3739.28
70	4900	343000	8.3666	4.1213	1.84510	14.2857	219.91	3848.45
71	5041	357911	8.4261	4.1408	1.85126	14.0845	223.05	3959.19
72	5184	373248	8.4853	4.1602	1.85733	13.8889	226.19	4071.50
73	5329	389017	8.5440	4.1793	1.86332	13.6986	229.34	4185.39
74	5476	405224	8.6023	4.1983	1.86923	13.5135	232.48	4300.84
75	5625	421875	8.6603	4.2172	1.87506	13.3333	235.62	4417.86
76	5776	438976	8.7178	4.2358	1.88081	13.1579	238.76	4536.46
77	5929	456533	8.7750	4.2543	1.88649	12.9870	241.90	4656.63
78	6084	474552	8.8318	4.2727	1.89209	12.8205	245.04	4778.36
79	6241	493039	8.8882	4.2908	1.89763	12.6582	248.19	4901.67
80	6400	512000	8.9443	4.3089	1.90309	12.5000	251.33	5026.55
81	6561	531441	9.0000	4.3267	1.90849	12.3457	254.47	5153.00
82	6724	551368	9.0554	4.3445	1.91381	12.1951	257.61	5281.02
83	6889	571787	9.1104	4.3621	1.91908	12.0482	260.75	5410.61
84	7056	592704	9.1652	4.3795	1.92428	11.9048	263.89	5541.77
85	7225	614125	9.2195	4.3968	1.92942	11.7647	267.04	5674.50
86	7396	636056	9.2736	4.4140	1.93450	11.6279	270.18	5808.80
87	7569	658503	9.3274	4.4310	1.93952	11.4943	273.32	5944.68
88	7744	681472	9.3808	4.4480	1.94448	11.3636	276.46	6082.12
89	7921	704969	9.4340	4.4647	1.94939	11.2360	279.60	6221.14
90	8100	729000	9.4868	4.4814	1.95424	11.1111	282.74	6361.73
91	8281	753571	9.5394	4.4979	1.95904	10.9890	285.88	6503.88
92	8464	778688	9.5917	4.5144	1.96379	10.8696	289.03	6647.61
93	8649	804357	9.6437	4.5307	1.96848	10.7527	292.17	6792.91
94	8836	830584	9.6954	4.5468	1.97313	10.6383	295.31	6939.78
95	9025	857375	9.7468	4.5629	1.97772	10.5263	298.45	7088.22
96	9216	884736	9.7980	4.5789	1.98227	10.4167	301.59	7238.23
97	9409	912673	9.8489	4.5947	1.98677	10.3093	304.73	7389.81
98	9604	941192	9.8995	4.6104	1.99123	10.2041	307.88	7542.96
99	9801	970299	9.9499	4.6261	1.99564	10.1010	311.02	7697.69
100	10000	1000000	10.0000	4.6416	2.00000	10.0000	314.16	7853.98

PROPERTIES OF THE CIRCLE

Circumference of Circle of Diameter 1 = π = 3.14159265
Circumference of Circle = $2\pi r$
Diameter of Circle = Circumference x 0.31831
Diameter of Circle of equal periphery as square = side x 1.27324
Side of Square of equal periphery as circle = diameter x 0.78540
Diameter of Circle circumscribed about square = side x 1.41421
Side of Square inscribed in Circle = diameter x 0.70711



Arc, $a = \dfrac{\pi r A^\circ}{180} = 0.017453\, rA^\circ$

Angle, $A = \dfrac{180^\circ a}{\pi r} = 57.29578\,\dfrac{a}{r}$

Radius, $r = \dfrac{4b^2 + c^2}{8b}$ Diameter, $d = \dfrac{4b^2 + c^2}{4b}$

Chord, $c = 2\sqrt{2br - b^2} = 2r\sin\dfrac{A^\circ}{2}$

Rise, $b = r - \dfrac{1}{2}\sqrt{4r^2 - c^2} = \dfrac{c}{2}\tan\dfrac{A^\circ}{4} = 2r\sin^2\dfrac{A}{4}$

Rise, $b = r + y - \sqrt{r^2 - x^2}$, $y = b - r + \sqrt{r^2 - x^2}$, $x = \sqrt{r^2 - (r + y - b^2)}$

$\pi = 3.14159265$, log = 0.4971499

$\dfrac{1}{\pi} = 0.3183099$, log = 9.5028501-10

$\pi^2 = 9.8696044$, log = 0.9942997

$\dfrac{1}{\pi^2} = 0.1013212$, log = 9.0057003-10

$\sqrt{\pi} = 1.7724539$, log = 0.2485749

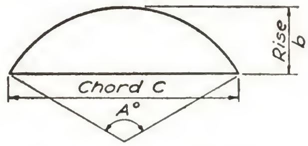
$\sqrt{\dfrac{1}{\pi}} = 0.5641896$, log = 9.7514251-10

$\dfrac{\pi}{180} = 0.0174533$, log = 8.2418774-10

$\dfrac{180}{\pi} = 57.2957795$, log = 1.7581226

AREAS OF CIRCULAR SEGMENTS

For Ratios of Rise and Chord



Area = b x C x coefficient

A°	Coefficient	b/C	A°	Coefficient	b/C	A°	Coefficient	b/C	A°	Coefficient	b/C
1	.6667	.0022	46	.6722	.1017	91	.6895	.2097	136	.7239	.3373
2	.6667	.0044	47	.6724	.1040	92	.6901	.2122	137	.7249	.3404
3	.6667	.0066	48	.6727	.1063	93	.6906	.2148	138	.7260	.3436
4	.6667	.0087	49	.6729	.1086	94	.6912	.2174	139	.7270	.3469
5	.6667	.0109	50	.6732	.1109	95	.6918	.2200	140	.7281	.3501
6	.6667	.0131	51	.6734	.1131	96	.6924	.2226	141	.7292	.3534
7	.6668	.0153	52	.6737	.1154	97	.6930	.2252	142	.7303	.3567
8	.6668	.0175	53	.6740	.1177	98	.6936	.2279	143	.7314	.3600
9	.6669	.0197	54	.6743	.1200	99	.6942	.2305	144	.7325	.3633
10	.6670	.0218	55	.6746	.1224	100	.6948	.2332	145	.7336	.3666
11	.6670	.0240	56	.6749	.1247	101	.6954	.2358	146	.7348	.3700
12	.6671	.0262	57	.6752	.1270	102	.6961	.2385	147	.7360	.3734
13	.6672	.0284	58	.6755	.1293	103	.6967	.2412	148	.7372	.3768
14	.6672	.0306	59	.6758	.1316	104	.6974	.2439	149	.7384	.3802
15	.6673	.0328	60	.6761	.1340	105	.6980	.2466	150	.7396	.3837
16	.6674	.0350	61	.6764	.1363	106	.6987	.2493	151	.7408	.3871
17	.6674	.0372	62	.6768	.1387	107	.6994	.2520	152	.7421	.3906
18	.6675	.0394	63	.6771	.1410	108	.7001	.2548	153	.7434	.3942
19	.6676	.0416	64	.6775	.1434	109	.7008	.2575	154	.7447	.3977
20	.6677	.0437	65	.6779	.1457	110	.7015	.2603	155	.7460	.4013
21	.6678	.0459	66	.6782	.1481	111	.7022	.2631	156	.7473	.4049
22	.6679	.0481	67	.6786	.1505	112	.7030	.2659	157	.7486	.4085
23	.6680	.0504	68	.6790	.1529	113	.7037	.2687	158	.7500	.4122
24	.6681	.0526	69	.6794	.1553	114	.7045	.2715	159	.7514	.4159
25	.6682	.0548	70	.6797	.1577	115	.7052	.2743	160	.7528	.4196
26	.6684	.0570	71	.6801	.1601	116	.7060	.2772	161	.7542	.4233
27	.6685	.0592	72	.6805	.1625	117	.7068	.2800	162	.7557	.4270
28	.6687	.0614	73	.6809	.1649	118	.7076	.2829	163	.7571	.4308
29	.6688	.0636	74	.6814	.1673	119	.7084	.2858	164	.7586	.4346
30	.6690	.0658	75	.6818	.1697	120	.7092	.2887	165	.7601	.4385
31	.6691	.0681	76	.6822	.1722	121	.7100	.2916	166	.7616	.4424
32	.6693	.0703	77	.6826	.1746	122	.7109	.2945	167	.7632	.4463
33	.6694	.0725	78	.6831	.1771	123	.7117	.2975	168	.7648	.4502
34	.6696	.0747	79	.6835	.1795	124	.7126	.3004	169	.7664	.4542
35	.6698	.0770	80	.6840	.1820	125	.7134	.3034	170	.7680	.4582
36	.6700	.0792	81	.6844	.1845	126	.7143	.3064	171	.7696	.4622
37	.6702	.0814	82	.6849	.1869	127	.7152	.3094	172	.7712	.4663
38	.6704	.0837	83	.6854	.1894	128	.7161	.3124	173	.7729	.4704
39	.6706	.0859	84	.6859	.1919	129	.7170	.3155	174	.7746	.4745
40	.6708	.0882	85	.6864	.1944	130	.7180	.3185	175	.7763	.4787
41	.6710	.0904	86	.6869	.1970	131	.7189	.3216	176	.7781	.4828
42	.6712	.0927	87	.6874	.1995	132	.7199	.3247	177	.7799	.4871
43	.6714	.0949	88	.6879	.2020	133	.7209	.3278	178	.7817	.4914
44	.6717	.0972	89	.6884	.2046	134	.7219	.3309	179	.7835	.4957
45	.6719	.0995	90	.6890	.2071	135	.7229	.3341	180	.7854	.5000

WEIGHTS AND MEASURES
UNITED STATES SYSTEM

Linear Measure

Inches		Feet		Yards		Rods		Furlongs		Miles
1.0	=	0.08333	=	0.02778	=	0.0050505	=	0.00012626	=	0.00001578
12.0	=	1.0	=	0.33333	=	0.0606061	=	0.000151515	=	0.00018939
36.0	=	3.0	=	1.0	=	0.1818182	=	0.00454545	=	0.00056818
198.0	=	16.5	=	5.5	=	1.0	=	0.025	=	0.003125
7920.0	=	660.0	=	220.0	=	40.0	=	1.0	=	0.125
63360.0	=	5280.0	=	1760.0	=	320.0	=	8.0	=	1.0

Square and Land Measure

Sq inches		Square feet		Square yards		Sq rods		Acres		Sq miles
1.0	=	0.006944	=	0.000772						
144.0	=	1.0	=	0.111111						
1296.0	=	9.0	=	1.0	=	0.03306	=	0.000207		
39204.0	=	272.25	=	30.25	=	1.0	=	0.00625	=	0.0000098
		43560.0	=	4840.0	=	160.0	=	1.0	=	0.0015625
				3097600.0	=	102400.0	=	640.0	=	1.0

Liquid Measure

Gills		Pints		Quarts		U. S. Gallons		Cubic feet
1.0	=	0.25	=	0.125	=	0.03125	=	0.00418
4.0	=	1.0	=	0.5	=	0.125	=	0.01671
8.0	=	2.0	=	1.0	=	0.250	=	0.03342
32.0	=	8.0	=	4.0	=	1.0	=	0.1337
						7.48052	=	1.0

Dry Measure

Pints		Quarts		Pecks		Cubic feet		Bushels
1.0	=	0.5	=	0.0625	=	0.01945	=	0.01563
2.0	=	1.0	=	0.125	=	0.03891	=	0.03125
16.0	=	8.0	=	1.0	=	0.31112	=	0.25
51.42627	=	25.71314	=	3.21414	=	1.0	=	0.80354
64.0	=	32.0	=	4.0	=	1.2445	=	1.0

Avoirdupois Weights

Grains		Drams		Ounces		Pounds		Tons
1.0	=	0.03657	=	0.002286	=	0.000143	=	0.0000000714
27.34375	=	1.0	=	0.0625	=	0.003906	=	0.00000195
437.5	=	16.0	=	1.0	=	0.0625	=	0.00003125
7000.0	=	256.0	=	16.0	=	1.0	=	0.0005
14000000.0	=	512000.0	=	32000.0	=	2000.0	=	1.0

CONVERSION FACTORS

Multiplying	By	Gives
acres	0.404687	hectares
"	4.04687×10^{-3}	square kilometers
ares	1076.39	square feet
board feet	144 sq in. × 1 in.	cubic inches
" "	0.0833	cubic feet
centimeters	3.28083×10^{-2}	feet
"	0.3937	inches
cubic centimeters	3.53145×10^{-5}	cubic feet
" "	6.102×10^{-2}	cubic inches
cubic feet	2.8317×10^{4}	cubic centimeters
" "	2.8317×10^{-2}	cubic meters
" "	6.22905	gallons, British Imperial
" "	28.3170	liters
" "	2.38095×10^{-2}	tons, British Shipping
" "	0.025	tons, U. S. Shipping
cubic inches	16.38716	cubic centimeters
cubic meters	35.3145	cubic feet
" "	1.30794	cubic yards
cubic yards	0.764559	cubic meters
degrees, angular	0.0174533	radians
degrees, Fahrenheit (less 32° F)	0.5556	degrees, Centigrade
" Centigrade	1.8	degrees, Fahrenheit (less 32° F)
foot pounds	0.13826	kilogram meters
feet	30.4801	centimeters
"	0.304801	meters
"	304.801	millimeters
"	1.64468×10^{-4}	miles, nautical
gallons, British Imperial	0.160538	cubic feet
" " "	1.20091	gallons, U. S.
" " "	4.54596	liters
gallons, U. S.	0.832702	gallons, British Imperial
" "	0.13368	cubic feet
" "	231.	cubic inches
" "	3.78543	liters
grams, metric	2.20462×10^{-3}	pounds, avoirdupois
hectares	2.47104	acres
"	1.076387×10^{5}	square feet
"	3.86101×10^{-3}	square miles
inches	2.54001	centimeters
"	2.54001×10^{-2}	meters
"	25.4001	millimeters
kilograms	2.20462	pounds
"	9.84206×10^{-4}	long tons
"	1.10231×10^{-3}	short tons
kilogram meters	7.233	foot pounds
kilograms per meter	0.671972	pounds per foot
kilograms per square centimeter	14.2234	pounds per square inch
kilograms per square meter	0.204817	pounds per square foot
" " " "	9.14362×10^{-5}	long tons per square foot
kilograms per square millimeter	1422.34	pounds per square inch
" " " "	0.634973	long tons per square inch
kilograms per cubic meter	6.24283×10^{-2}	pounds per cubic foot
kilometers	0.62137	miles, statute
"	0.53959	miles, nautical

CONVERSION FACTORS

Multiplying	By	Gives
liters	0.219975	gallons, British Imperial
"	0.26417	gallons, U. S.
"	3.53145×10^{-2}	cubic feet
meters	3.28083	feet
"	39.37	inches
"	1.09361	yards
miles, statute	1.60935	kilometers
" "	0.8684	miles, nautical (knots)
miles, nautical (knots)	6080.204	feet
" " "	1.85325	kilometers
" " "	1.1516	miles, statute
millimeters	3.28083×10^{-3}	feet
"	3.937×10^{-2}	inches
pounds, avoirdupois	453.592	grams, metric
" "	0.453592	kilograms
" "	4.464×10^{-4}	tons, long
" "	4.53592×10^{-4}	tons, metric
pounds per foot	1.48816	kilograms per meter
pounds per square foot	4.88241	kilograms per square meter
pounds per square inch	7.031×10^{-2}	kilograms per square centimeter
" " " "	7.031×10^{-4}	kilograms per square millimeter
pounds per cubic foot	16.0184	kilograms per cubic meter
radians	57.29578	degrees, angular
square centimeters	0.1550	square inches
square feet	9.29034×10^{-4}	ares
" "	9.29034×10^{-6}	hectares
" "	0.0929034	square meters
square inches	6.45163	square centimeters
" "	645.163	square millimeters
square kilometers	247.104	acres
" "	0.3861	square miles
square meters	10.7639	square feet
" "	1.19599	square yards
square miles	259.0	hectares
" "	2.590	square kilometers
square millimeters	1.550×10^{-3}	square inches
square yards	0.83613	square meters
tons, long	1016.05	kilograms
" "	2240.	pounds
" "	1.01605	tons, metric
" "	1.120	tons, short
tons, long, per square foot	1.09366×10^{-4}	kilograms per square meter
tons, long, per square inch	1.57494	kilograms per square millimeter
tons, metric	2204.62	pounds
" "	0.98421	tons, long
" "	1.10231	tons, short
tons, short	907.185	kilograms
" "	0.892857	tons, long
" "	0.907185	tons, metric
tons, British Shipping	42.00	cubic feet
" " "	0.952381	tons, U. S. Shipping
tons, U. S. Shipping	40.00	cubic feet
" " "	1.050	tons, British Shipping
yards	0.914402	meters

DECIMAL EQUIVALENTS OF AN INCH AND OF A FOOT

Fractions of Inch or Foot		Inch Equivalents to Foot Fractions	Fractions of Inch or Foot		Inch Equivalents to Foot Fractions	Fractions of Inch or Foot		Inch Equivalents to Foot Fractions	Fractions of Inch or Foot		Inch Equivalents to Foot Fractions
	.0052	1/16		.2552	3-1/16		.5052	6-1/16		.7552	9-1/16
	.0104	1/8		.2604	3-1/8		.5104	6-1/8		.7604	9-1/8
1/64	.015625	3/16	17/64	.265625	3-3/16	33/64	.515625	6-3/16	49/64	.765625	9-3/16
	.0208	1/4		.2708	3-1/4		.5208	6-1/4		.7708	9-1/4
	.0260	5/16		.2760	3-5/16		.5260	6-5/16		.7760	9-5/16
1/32	.03125	3/8	9/32	.28125	3-3/8	17/32	.53125	6-3/8	25/32	.78125	9-3/8
	.0365	7/16		.2865	3-7/16		.5365	6-7/16		.7865	9-7/16
	.0417	1/2		.2917	3-1/2		.5417	6-1/2		.7917	9-1/2
3/64	.046875	9/16	19/64	.296875	3-9/16	35/64	.546875	6-9/16	51/64	.796875	9-9/16
	.0521	5/8		.3021	3-5/8		.5521	6-5/8		.8021	9-5/8
	.0573	11/16		.3073	3-11/16		.5573	6-11/16		.8073	9-11/16
1/16	.0625	3/4	5/16	.3125	3-3/4	9/16	.5625	6-3/4	13/16	.8125	9-3/4
	.0677	13/16		.3177	3-13/16		.5677	6-13/16		.8177	9-13/16
	.0729	7/8		.3229	3-7/8		.5729	6-7/8		.8229	9-7/8
5/64	.078125	15/16	21/64	.328125	3-15/16	37/64	.578125	6-15/16	53/64	.828125	9-15/16
	.0833	1		.3333	4		.5833	7		.8333	10
	.0885	1-1/16		.3385	4-1/16		.5885	7-1/16		.8385	10-1/16
3/32	.09375	1-1/8	11/32	.34375	4-1/8	19/32	.59375	7-1/8	27/32	.84375	10-1/8
	.0990	1-3/16		.3490	4-3/16		.5990	7-3/16		.8490	10-3/16
	.1042	1-1/4		.3542	4-1/4		.6042	7-1/4		.8542	10-1/4
7/64	.109375	1-5/16	23/64	.359375	4-5/16	39/64	.609375	7-5/16	55/64	.859375	10-5/16
	.1146	1-3/8		.3646	4-3/8		.6146	7-3/8		.8646	10-3/8
	.1198	1-7/16		.3698	4-7/16		.6198	7-7/16		.8698	10-7/16
1/8	.1250	1-1/2	3/8	.3750	4-1/2	5/8	.6250	7-1/2	7/8	.8750	10-1/2
	.1302	1-9/16		.3802	4-9/16		.6302	7-9/16		.8802	10-9/16
	.1354	1-5/8		.3854	4-5/8		.6354	7-5/8		.8854	10-5/8
9/64	.140625	1-11/16	25/64	.390625	4-11/16	41/64	.640625	7-11/16	57/64	.890625	10-11/16
	.1458	1-3/4		.3958	4-3/4		.6458	7-3/4		.8958	10-3/4
	.1510	1-13/16		.4010	4-13/16		.6510	7-13/16		.9010	10-13/16
5/32	.15625	1-7/8	13/32	.40625	4-7/8	21/32	.65625	7-7/8	29/32	.90625	10-7/8
	.1615	1-15/16		.4115	4-15/16		.6615	7-15/16		.9115	10-15/16
	.1667	2		.4167	5		.6667	8		.9167	11
11/64	.171875	2-1/16	27/64	.421875	5-1/16	43/64	.671875	8-1/16	59/64	.921875	11-1/16
	.1771	2-1/8		.4271	5-1/8		.6771	8-1/8		.9271	11-1/8
	.1823	2-3/16		.4323	5-3/16		.6823	8-3/16		.9323	11-3/16
3/16	.1875	2-1/4	7/16	.4375	5-1/4	11/16	.6875	8-1/4	15/16	.9375	11-1/4
	.1927	2-5/16		.4427	5-5/16		.6927	8-5/16		.9427	11-5/16
	.1979	2-3/8		.4479	5-3/8		.6979	8-3/8		.9479	11-3/8
13/64	.203125	2-7/16	29/64	.453125	5-7/16	45/64	.703125	8-7/16	61/64	.953125	11-7/16
	.2083	2-1/2		.4583	5-1/2		.7083	8-1/2		.9583	11-1/2
	.2135	2-9/16		.4635	5-9/16		.7135	8-9/16		.9635	11-9/16
7/32	.21875	2-5/8	15/32	.46875	5-5/8	23/32	.71875	8-5/8	31/32	.96875	11-5/8
	.2240	2-11/16		.4740	5-11/16		.7240	8-11/16		.9740	11-11/16
	.2292	2-3/4		.4792	5-3/4		.7292	8-3/4		.9792	11-3/4
15/64	.234375	2-13/16	31/64	.484375	5-13/16	47/64	.734375	8-13/16	63/64	.984375	11-13/16
	.2396	2-7/8		.4896	5-7/8		.7396	8-7/8		.9896	11-7/8
	.2448	2-15/16		.4948	5-15/16		.7448	8-15/16		.9948	11-15/16
1/4	.2500	3	1/2	.5000	6	3/4	.7500	9	1	1.0000	12

INDEX